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A TREATISE
ON
CONCRETE
PLAIN AND REINFORCED

MATERIALS, CONSTRUCTION, AND DESIGN OF
CONCRETE AND REINFORCED CONCRETE

WITH CHAPTERS BY
R. FERET, WILLIAM B. FULLER & SPENCER B. NEWBERRY

BY
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PREFACE

This treatise is designed for practicing engineers and contractors, and also for a text and reference book on concrete for engineering students.

To broaden the scope of the work and avoid personal inaccuracies, each chapter has been submitted for critcism to at least one, and, in some cases, to three or four specialists in the particular line treated. We have aimed to refer by name to all authorities quoted, and where the data is taken from books or periodicals, to give the original publication, so that each subject may be investigated further. Proof clippings have also been submitted for approval to those whose names are mentioned. Numerous cross references will be found as well as many repetitions, inserted for the purpose of emphasizing important facts.

The chapters are arranged for convenience in reference, and therefore are not always in logical order.

The Concrete Data in Chapter I presents a list of definitions of words and terms relating distinctively to cement and concrete; a summary of the most important facts and conclusions, with references to the pages discussing them; data on concrete labor, and conversion ratios.

The Elementary Outline of the Process of Concreting, Chapter II, is designed, not for the civil engineer, but for those seeking simple directions as to the exact procedure in laying a small quantity of concrete. Most of the subjects there treated are discussed at length in subsequent chapters.

The Specifications for Cement in Chapter III include the latest recommendations of committees of our national societies, with incidental changes to adapt them for direct use in purchase specifications. The Concrete Specifications have been prepared by the authors to represent standard practice. Specifications for First-class or High Steel, drawn up by Mr. Taylor, are, we believe, the first recommendations which have been made to safely adapt this important material to reinforced concrete construction.

In Chapter IV the Choice of Cement is considered in an elementary fashion, which will serve as a guide to the constructor. Classification of Cements, Chapter V, distinguishes the various cements and limes manufactured in the United States and Europe.

Mr. Spencer B. Newberry, an international authority on the subject treated, has very kindly written for us Chapter VI on the Chemistry of Hydraulic Cement, discussing this complex subject in such a clear and practical manner that it will be of interest not only to the scientist, but also to the general reader and to the cement manufacturer. Mr. Newberry has also criticised Chapter V.

Chapters VII and VIII give the latest information on the testing of cement. Chapter IX presents practical rules for selecting sand for mortar, and the effect of different sands and of foreign ingredients upon its quality. Characteristics of the Aggregate are further treated, and practical data in regard to it are given in Chapter X.

The subject of Proportioning Concrete has been treated, at our request, by Mr. William B. Fuller, the concrete expert, and his practical use of mechanical analysis is fully discussed.

The tables of Quantities of Materials for Concrete and Mortar, in Chapter XII, and the diagram of curves, will be found useful in estimating materials.

The Strength of Concrete, Chapter XIII, is taken up from a practical standpoint so that the data may be directly employed in design.

The theory and design of reinforced concrete are as yet in an elementary stage, but the rules and tables in Chapter XIV represent the most advanced knowledge on the subject.

Practical methods of Mixing and Laying Concrete are treated in Chapters XV, XVI, and XVII.

Mr. René Feret, of Boulogne-sur-Mer, France, whose extended researches enable him to speak with authority, has kindly written for us Chapter XVIII, entitled The Effect of Sea Water.

Chapters XIX, XX, and XXI, on Freezing, Water-Tightness, and Fire and Rust Protection, are of practical interest to the contracting engineer.

Plain and Reinforced Concrete Structures are treated in as much detail as space permits in Chapters XXII to XXVII inclusive. The designs are taken mostly from original drawings redrawn by the authors. They have been selected, not as extraordinary productions, but because the data in regard to them may be of use in designing similar structures.

Methods of Cement Manufacture in its modern types are described in detail in Chapter XXVIII.

The References in Chapter XXIX will be found especially valuable to one pursuing more extended investigations than can be presented in a volume of this size.

They have been selected from the large number contained in the authors' index, as those which it may be to the advantage of the reader to consult.

The articles are usually described by their subject-matter rather than by their titles verbatim.

Appendix I gives the method of chemically analyzing cement and cement materials according to the recommendations of the American Chemical Society.

Additional formulas for reinforced concrete beams, too complicated for insertion in the body of the book, are given in Appendix II, these having been kindly compiled by Prof. Frank P. McKibben for this treatise.

The authors desire to express their sincere appreciation of the various kindnesses extended to them while compiling the work. It has been necessary, because of the lack of authoritative information on many fundamental questions, not only to conduct numerous original investigations, but also to correspond with the most prominient engineers in this country, and with experts in England, France, and Austria.

Mr. Feret, besides writing the chapter on The Effect of Sea Water, has kindly criticised Chapter IX, and made numerous suggestions which have been incorporated.

Mr. Fuller has examined and criticised all the chapters on practical construction, and Prof. McKibben has rendered material assistance in the line of investigations and criticisms relating to the theories of reinforced concrete.

The authors are indebted to many gentlemen for careful criticism of chapters or portions of chapters, for drawings, or for replies to questions, and take this opportunity to express their sincere appreciation of all such assistance. Among those to whom especial acknowledgment is due are the following:

Messrs. Earle C. Bacon, David B. Butler (England), Harry T. Buttolph, Howard A. Carson, Edwin C. Eckel, William E. Foss, George B. Francis, John R. Freeman, Charles S. Gowen, Allen Hazen, Rudolph Hering, James E. Howard, Richard L. Humphrey, A. L. Johnson, George A. Kimball, Alfred Noble, William Barclay Parsons, Henry H. Quimby, George W. Rafter, Ernest L. Ransome, Clifford Richardson, Thomas F. Richardson, A. E. Schutté, W. Purves Taylor, Edwin Thacher, Leonard C. Wason, George S. Webster, Robert Spurr Weston, Joseph R. Worcester; and Professors Ira O. Baker, Lewis J. Johnson, Edgar B. Kay, Gaetano Lanza, Charles L. Norton, Charles M. Spofford, George F. Swain, Arthur N. Talbot.

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Machine Works, Fairbanks Co., Falkenau-Sinclair Machine Co., Farrel Foundry and Machine Co., Iroquois Iron Works, Kent Mill Co., Link-Belt Engineering Co., McKelvey Concrete Machinery Co., W. F. Mosher & Son, Tinius Olsen and Co., Philadelphia Pneumatic Tool Co , Thos. Prosser and Son, Ransome Concrete Machinery Co., Riehlé Bros. Testing Machine Co., Robins Conveying Belt Co., Sherburne and Co., T. L. Smith, Henry Troemner, Tucker and Vinton.

FREDERICK W. TAYLOR.
SANFORD E. THOMPSON.

February, 1905.

The writer wishes to state that the investigation and study necessary for the writing of this book were done by his colleague, Mr. Thompson, and desires that full credit for this should be given to him.

FREDERICK W. TAYLOR.

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A Treatise on Concrete

CHAPTER I

CONCRETE DATA

DEFINITIONS

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Aggregate is the inert material, such as sand, broken stone, etc., with which the cement or other adhesive material is mixed to form concrete or mortar. The term is sometimes erroneously applied to the coarse material, such as broken stone, only.

Akron Cement is a Natural cement from the vicinity of Akron, N. Y. 49

Beton is the French word for concrete.

Beton-Coignet is a mixture of hydraulic lime, cement, and sand 42

Concrete* is an artificial stone made by mixing cement, or some similar material — which after mixing with water will set or harden so as to adhere to inert material, — and an aggregate composed of hard, inert particles of varying size, such as a combination of sand or broken stone screenings, with gravel, broken stone, cinders, broken brick, or other coarse material.

Concrete Rubble is masonry of large stones, usually of derrick size, with joints of concrete instead of mortar 387

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Hydraulic Lime contains lime and clay in such proportions that it hardens under water 52

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*Also applied to mixtures of an aggregate with a material such as asphalt — which liquefies on application of heat.

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WEIGHTS AND VOLUMES

Portland Cement weighs per barrel, net	376	lb.	29
“ “ “ “ bag “	94	“	29
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“ “ “ “ bag, net	94	“	31
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Soundness. A sound cement will not go to pieces on the work. The test is therefore of greatest importance, and is often the only one necessary. Take about ½ pound, or one cupful, of Portland cement and mix by kneading 1½ minutes with sufficient water to form a paste of a consistency like putty. Press portions of the paste on to 3 pieces of window glass 4 inches square, so as to make 3 pats each about 3 inches in diameter and ½ inch thick at center tapering to a thin edge, and place in moist air for 24 hours. Then keep one pat in air at moderate temperature (about 60° or 70° Fahr.) for 28 days, keep second pat in water for 28 days, and place third pat in loosely closed vessel over boiling water and keep there for five hours. Reject cement if any pats show radial cracks or curl or crumble. The air

pat should not change color. Portland cements may be accepted on the steam test alone if time is limited. Natural cements should be subjected to water and air but not to steam. (See p. 79.)

Fineness. The finer the cement of a certain class the higher is its value. Sift 5 ounces of dry cement containing no lumps through a sieve about 6 to 8 inches diameter with 100 meshes per linear inch. Not more than ½ ounce of either Portland or Natural cement should remain on sieve. To compare quality of two brands otherwise similar, sift through a 200-mesh sieve and choose the finer cement. (See p. 67.)

Setting. A quick-setting cement is difficult to handle on the work and a too slow setting cement delays removal of forms. If a Vicat needle cannot be obtained for testing, use the Gillmore needles, — two steel rods, one, one-twelfth inch diameter at its end, loaded to weigh ¼ pound, the other, one-twenty-fourth inch diameter loaded to weigh 1 pound. A pat of pure Portland cement paste made like the soundness pat must not be able to support the weight of the lighter needle until 30 minutes after mixing, and must support the heavier needle in less than 10 hours. A paste or mortar or concrete has reached its final set when it will support a pressure of the thumb without indenting. (See p. 70.)

Purity. "Provide a glass-stoppered bottle of muriatic acid, two shallow white bowls or two ½-inch by 6-inch test tubes, a glass rod, and a pair of rubber gloves. Put in a bowl or a tube as much cement as can be taken on a nickel 5-cent piece; moisten it with half a teaspoonful of water; cover with clear muriatic acid poured slowly upon the cement while stirring it with the glass rod. Pure Portland cement will effervesce slightly, and will give off some pungent gas and will gradually form a bright yellow jelly without any sediment. Powdered limestone or powdered cement-rock mixed with the pure cement will cause a violent effervescence, the acid boiling and giving off strong fumes until all the carbonate of lime has been consumed, when the bright yellow jelly will form. Powdered sand or quartz or silica mixed with cement will produce no other effect than to remain undissolved as a sediment at the bottom of the yellow jelly. Reject cement which has either of these adulterants."* (See p. 65.)

Tensile Strength. The tensile test is frequently unnecessary with a standard brand of cement employed in ordinary construction. Neat Portland cement should test at least 500 pounds in 7 days and 600 pounds in 28 days. Mixed with three parts standard sand by weight, it should test at least 150 pounds in 7 days and 200 pounds in 28 days. (See p. 30.)

*Judson's City Roads and Pavements, 1902.

Specific Gravity. The test requires delicate apparatus and is seldom necessary. Specific gravity of Portland cement should exceed 3.1. (See p. 30.)

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STRENGTH OF CONCRETE AND MORTAR

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DATA ON HANDLING CONCRETE

Average load of broken stone or gravel for wood wheelbarrow	2.4	cu. ft.
“ “ “ sand for wood wheelbarrow	2.5	“ “
Large load of broken stone or gravel for iron wheelbarrow on short haul in concrete work	3.0	“ “
Large load of sand for iron wheelbarrow on short haul in concrete work	3.5	“ “
Average load of ordinary concrete* for iron wheelbarrow	1.9	“ “
Large “ “ “ “ “ “ “	2.2	“ “
Number of shovelfuls of concrete per barrow in average load	13	
“ “ “ “ “ “ “ “ large “	15	
Average net time of one man filling wheelbarrow with concrete,	1⅓	min.
Quick “ “ “ “ “ “ “ “ “	1	“
Average quantity concrete* mixed, wheeled 50 ft., and rammed, per man, per day of 10 hours†	2.2	cu. yd.
Large quantity concrete* mixed, wheeled 50 ft. and rammed, per man, per day of 10 hours†	3	“ “
Average quantity concrete* laid as above with a gang of 15 men per day of 10 hours†	33	“ “
Large quantity concrete* laid as above with a gang of 15 men per day of 10 hours†	47	“ “
Approximate average quantity of concrete* leveled and rammed in 6-inch layers, per man, per day of 10 hours	11	“ “
Approximate large quantity of concrete* leveled and rammed in 6-inch layers, per man, per day of 10 hours	16	“ “
Approximate average surface of rough braced plank form built and removed by one carpenter per day of 10 hours	25	sq. “

CHANGING FOREIGN TO AMERICAN MEASURES

To convert values of kilograms per square centimeter to pounds per square inch, multiply the former by 14.2 (more exactly 14.2234).

To convert values of pounds per square inch to kilograms per square centimeter, multiply the former by 0.07 (more exactly, 0.07031).

*All measurements of concrete are reduced to terms of quantity in place after ramming.

†Note that the leveling and ramming, but not the labor on form, are included in this item.

To convert values of pounds per square inch to tons (2,000 lb.) per square foot, divide the former by 14 (more exactly 13.89).

To convert Centigrade to Fahrenheit temperatures, multiply the former by 1.8 and add 32° to the product.

To convert Fahrenheit to Centigrade temperature, deduct 32° from the former and divide by 1.8.

One millimeter = 0.0394 inch
One centimeter = 0.3937 "
One meter = 39.37 inches or 3.281 feet
One square centimeter = 0.155 square inch
One " meter = 10.764 square feet or 1.196 square yards
One cubic centimeter = 0.061 cubic inch
One " meter = 35.31 cubic feet, or 1.308 cubic yards
One liter = 61.02 cubic inches or 0.0353 cubic foot, or 1.057 U. S. liquid quarts or 0.2642 U. S. liquid gallon
One gram = 0.0353 avoirdupois ounce
500 grams = 1.1 pounds avoirdupois
One kilogram = 2.2046 pounds avoirdupois
One tonne or metric ton = 2204.62 pounds or 1.1023 tons (of 2,000 lb.)
One English penny = \$0.0203
One " shilling = \$0.2433
One " pound = \$4.8665
One French franc = \$0.193
One German mark = \$0.238

CHAPTER II

ELEMENTARY OUTLINE OF THE PROCESS OF CONCRETING

This chapter is not written for experienced civil engineers and contractors, nor for those who desire to make a scientific study of methods and principles. On the contrary, it is merely an elementary outline, indicating to the inexperienced the various steps which must be taken with this class of masonry. In subsequent chapters the various divisions of the subject are treated in detail.

The question as to whether concrete is preferable to some other form of masonry may often resolve itself into a question of cost. The cost, in turn, is dependent upon the character of the structure, the rate of labor and the price of the various materials entering into the work. Portland cement concrete has been laid in large masses at as low a price as $3 per cubic yard, while for thin walls built under disadvantageous conditions the cost of constructing molds may cause it to run as high as $30 per cubic yard, and in the case of ornamental work even above this. Before estimating the cost in any case, the materials must be chosen and the relative proportions of the ingredients determined from a consideration of the design of the structure.

WHERE CONCRETE MAY BE USED

By far the largest proportion of Portland cement concrete is laid in heavy foundation work and in other structures, such as tunnels and subways, below the surface of the ground. It is peculiarly adapted for foundations of engines or machinery, heavy walls, piers, etc. In the former the concrete is often carried all the way up to the base of the engine or machine, instead of being topped with brick or stone. It is widely used for sidewalks or floors upon the ground level, and for suspended floors. When suitably reinforced with steel, it furnishes probably the most economical and effective material for fire-proof construction. Its use for walls of buildings is largely increasing, but on account of the very indefinite time required in the building and moving of forms the cost may largely exceed the original estimate unless the builder is experienced in this class of work. Under favorable conditions, however, a 6-inch wall of concrete will cost no more, and usually less, than a 12-inch wall of brick work, and will be

stronger, more durable, and fire-proof. The strength of concrete columns and beams is readily calculated by means of formulas.

Concrete is destined to be used to a large extent in the construction of tanks and vats for holding various liquids which attack wood and iron. Their construction is comparatively simple, but the work must be carefully performed if the result is to be permanent and satisfactory. Concrete is especially suitable for all kinds of arches, because the stresses therein are chiefly compressive. Other classes of work for which concrete is largely employed are dams, retaining walls, penstocks, bridges, abutments, sewer and water conduits, and reservoirs. For ornamental work developments are constantly being made, and it is noteworthy that concrete or mortar can be cast in molds in a somewhat similar manner to that in which plaster of Paris is run for interior decoration.

SELECTION OF MATERIALS

Concrete is ordinarily composed of cement, sand, gravel or crushed stone, or both, and water. The selection of each of these materials is largely dependent upon local conditions, and no unalterable rule can be laid down in regard to it, but certain general conditions may serve as a guide to the inexperienced.

Cement. It is a wise rule to use Portland cement for nearly all classes of concrete, and the remarks in this chapter are based entirely upon this material. Portland cement is more uniform and therefore more reliable, while its strength is so much higher than Natural cement that by mixing it with larger proportions of sand and stone, properly graded, it will usually yield better results at less cost than Natural cement.

If the job is small and unimportant, it is generally safe to select in the market a brand of Portland cement of American manufacture which has a first-class reputation, and to use it without testing. As a precaution, however, it is usually advisable that samples from a few of the packages of every shipment be tested for soundness. This can be done after a little practice with scarcely any apparatus. (See p. 79.) For very important concrete construction complete specifications should be prepared before purchasing the cement, and a small laboratory established for conducting tests to determine whether it is fulfilling the requirements. (See p. 28.)

Aggregate. The sand and broken stone or gravel are termed the aggregate. The sand should be clean. One may obtain some idea of its cleanliness by placing it in the palm of one hand and rubbing it with the fingers of the other. If the sand is dirty, it will badly discolor the palm.

If the use of dirty sand is unavoidable, its effect upon the strength of the mortar should be investigated. Preference should be given to sand containing a mixture of coarse and fine grains. Extremely fine sand can be used alone, but it makes a weaker mortar than either coarse sand alone or a mixture of coarse and fine sand.

Either crushed stone or clean gravel, or both, is suitable for the coarse material of the aggregate. It is chiefly a question of which can be delivered upon the work at the least cost. If the gravel is chosen greater uniformity is attained by screening it over, say a ⅜-inch mesh screen, and then remixing the sand which falls through the screen with the coarser gravel in definite proportions, than by taking the run of the bank. If the gravel is dirty or clayey it should be washed with a hose, a little at a time, before it is shoveled on to the mixing platform.

Broken stone, if selected, may be used unscreened as it comes from the crusher, although it is preferable to screen out the dust and to use the latter as a portion of the sand. The maximum size is usually limited to 2½ inches. A smaller size than this, say one inch, will give, with less care, a finer surface. In a thick wall large sound stones may be placed by hand or derrick without detriment to the work, providing the consistency of the concrete is thin enough to properly imbed them.

PROPORTIONS

Accurate methods of proportioning the cement and aggregate in concrete are discussed in chapter XI, page 183, and if a large or very important mass is under consideration, or if the work must be water-tight, the correct proportioning requires more careful consideration than can be given it in this chapter. The method often adopted of pouring water into the coarser material to determine the percentage of voids, and thus finding the quantity of sand to use for filling them, is apt to be misleading, because so much depends upon the compactness of the stone, due to the method of handling it — that is, whether placed quietly, dropped, thrown, or shaken down — and because in the majority of cases the sand contains many grains so large that they will not enter the smaller voids of the coarser material. In a small job it is sufficiently accurate to select the proportion of cement to sand which will give the required strength to the concrete, and then use twice as much gravel or broken stone as sand. In figuring the capacities of the measures for the sand and stone it must be remembered that a barrel of Portland cement weighs 376 pounds, not including the barrel, and a bag of Portland cement 94 pounds, and we may assume for convenience

that a cement barrel holds 3.8 cubic feet. This is a fair average measurement of a heaped barrel, or a barrel with both heads removed — a convenient measure for sand.

As a rough guide to the selection of materials for various classes of work; we may take four proportions which differ from each other simply in the relative quantity of cement:

A rich mixture, for reinforced engine or machine foundations subject to vibrations, for reinforced floors, beams and columns for heavy loading, tanks and other water-tight work, — proportions 1:2:4; that is, 1 barrel (4 bags) packed Portland cement (as it comes from the manufacturer) to 2 barrels (7.6 cubic feet) loose sand, to 4 barrels (15.2 cubic feet) loose gravel or broken stone;

A medium mixture, for ordinary machine foundations, thin foundation walls, building walls, arches, ordinary floors, sidewalks, and sewers, — proportions 1:2½:5, that is, 1 barrel (4 bags), packed Portland cement to 2½ barrels (9.5 cubic feet) loose sand, to 5 barrels (19 cubic feet) loose gravel or broken stone;

An ordinary mixture, for heavy walls, retaining walls, piers, and abutments, which are to be subjected to considerable strain, — proportions are 1:3:6, that is, 1 barrel (4 bags), packed Portland cement, to 3 barrels (11.4 cubic feet) loose sand, to 6 barrels (22.8 cubic feet) loose gravel or broken stone;

A lean mixture, for unimportant work in masses where the concrete is subjected to plain compressive strain, as in large foundations supporting a stationary load or backing for stone masonry, — proportions are 1:4:8, that is, 1 barrel (4 bags) packed Portland cement, to 4 barrels (15.2 cubic feet) loose sand, to 8 barrels (30.4 cubic feet) loose gravel or broken stone.

The above specifications are based upon fair average practice. If the aggregate is carefully graded and the proportions are scientifically fixed, smaller proportions of cement may be used for each class of work.

QUANTITIES OF MATERIAL

Inexperienced contractors have often lost money by assuming that the quantity of gravel plus the quantity of sand required will be equivalent to the volume of the finished concrete — that is, that 7½ cubic yards of concrete in the proportions of 1:2½:5 will require 2½ cubic yards of sand and 5 cubic yards of gravel. This is absolutely wrong, since the grains of sand fill, to a certain extent, the spaces between the larger pebbles. It is incorrect, on the other hand, to figure a quantity of gravel equal to the total

volume of the concrete, because the introduction of the mortar, which is always in excess of the actual voids, swells the bulk.

If gravel or stone having particles of uniform size is used it must be recognized that the work will cost from 5 to 10 per cent. more, on account of the additional quantity of material required to make a given volume of concrete. In measuring the gravel or stone before mixing there will be less solid matter in a measure, and consequently more sand and cement will be necessary to fill the spaces between the stones. This fact, which is often overlooked even by experienced men, is illustrated in a somewhat exaggerated fashion in Figs. 1 and 2. Here Fig. 1 illustrates the measurement of the dry materials and the mixture produced therefrom when the stone has been screened to one uniform size, while Fig. 2 shows the dry materials and the mixture when the stone is what is termed "crusher run" — that is, of varying sizes as it comes from the crusher.

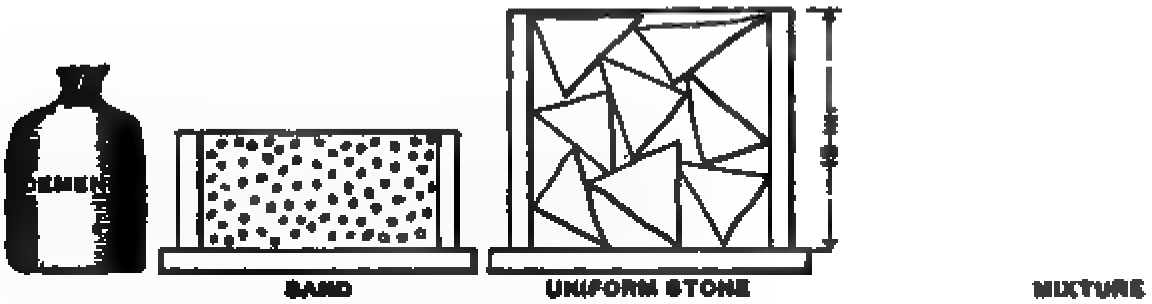

Fig. 1.—Diagram illustrating measurement of Dry Materials and the Mixture when Broken Stone is of uniform size. (*See p.* 15.)

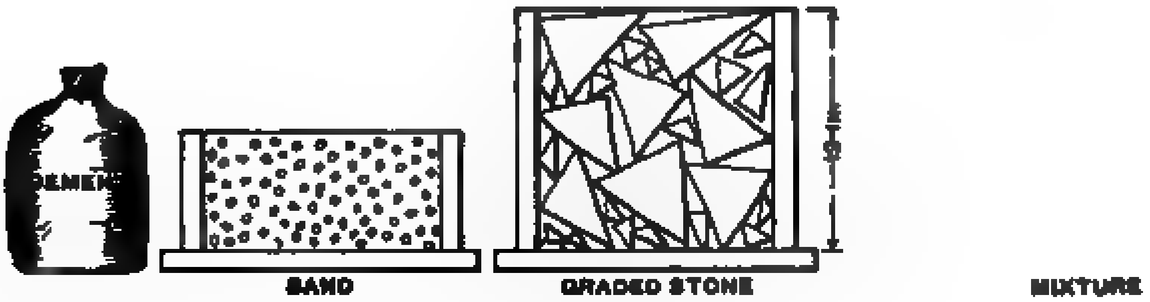

Fig. 2.—Dry Materials and Mixture when the Stone is of varying sizes. (*See p.* 15.)

It is obvious at a glance that the uniform stone measured in Fig. 1 contains less solid stone than the graded stone measured in Fig. 2. The spaces between the stones in the first case are very nearly equal to the volume of

the solid particles, and as the measure of the sand is one-half that of the stone, and the particles of cement fill the voids in the sand, this sand and cement mixes in between the stones, filling the spaces or voids, and resulting in a mixture but very slightly greater in volume than the stone alone. In the second case, Fig. 2, the spaces between the large stones in the stone measure are filled with graded smaller stones, so that there is a much smaller volume of spaces or voids. Hence, when the sand and cement, which are identical with that in Fig. 1, are mixed with it the volume of mixture becomes considerably larger than the original bulk of the stone. Consequently, if we start with definite proportions of materials, more concrete will be made with graded stone — such as "crusher run" broken stone, or gravel containing various sizes, ranging, say, from ¼ inch up to 2 inches — than if the stone has been screened to uniform size. If, on the other hand, the proportions of the materials are changed on account of the fewer voids in the mixed stone, and less sand and cement are used, a saving in these materials results.

Fuller's Rule For Quantities—The simplest rule for determining the quantities of materials for a cubic yard of concrete is one devised by William B. Fuller. Expressed in words, it is as follows:

Divide 11 by the sum of the parts of all the ingredients, and the quotient will be the number of barrels of Portland cement required for 1 cubic yard of concrete. The number of barrels of cement thus found, multiplied respectively by the "parts" of sand and stone, will give the number of barrels of each required for 1 cubic yard of concrete, and multiplying these values by 3.8 (the number of cubic feet in a barrel), and dividing by 27 (the number of cubic feet in a cubic yard), will give the quantities of sand and stone, in fractions of a cubic yard, needed for 1 cubic yard of concrete.

To express this rule in the shape of formulas:

Let

c = number of parts cement;

s = number of parts sand;

g = number of parts gravel or broken stone.

Then

$$\frac{11}{c+s+g} = P = \text{number of barrels Portland cement required for one cubic yard of concrete.}$$

$$P \times s \times \frac{3.8}{27} = \text{number of cubic yards of sand required for one cubic yard of concrete.}$$

$P \times g \times \frac{3.8}{27}$ = number of cubic yards of stone or gravel required for one cubic yard of concrete.

The following table is made up from Fuller's rule and represents fair averages of all classes of material. The first figure in each proportion represents the unit, or one barrel (4 bags), of packed Portland cement (weighing 376 pounds), the second figure, the number of barrels loose sand (3.8 cubic feet each) per barrel of cement, and the third figure, the number of barrels loose gravel or stone (of 3.8 cubic feet each) per barrel of cement:

Materials for One Cubic Yard of Concrete.

Proportions.	Cement, Barrels.	Sand, Cubic yards.	Gravel or stone, Cubic yards.
1:2:4	1.57	0.44	0.88
1:2½:5	1.29	0.45	0.91
1:3:6	1.10	0.46	0.93
1:4:8	0.85	0.48	0.96

If the coarse material is broken stone screened to uniform size it will, as is stated above, contain less solid matter in a given volume than an average stone, and about 5 per cent. must be added to the quantities of *all* the materials. If the coarse material contains a large variety of sizes so as to be quite dense, about 5 per cent. may be deducted from all of the quantities.

Example.— What materials will be required for six machine foundations, each 5 feet square at the bottom, 4 feet square at the top, and 8 feet high?

Answer. — Each pier contains 163 cubic feet, and the six piers therefore contain $\frac{6 \times 163}{27} = 36.2$ cubic yards. If we select proportions 1: 2½: 5, we find, multiplying the total volume by the quantities given in the table, that there will be required, in round numbers, 47 barrels packed cement, 16½ cubic yards loose sand, 33 cubic yards loose gravel.

TOOLS AND APPARATUS REQUIRED FOR CONCRETE WORK

The quantity of tools will, of course, vary with the size of the gang. The following schedule is based upon a small gang of eight or ten men, making concrete by hand:

Eight square pointed shovels, size No. 3, and such as illustrated in Fig. 3, page 18. (If a very wet mixture is used substitute small coal scoops.)

Three iron wheelbarrows, Fig. 4, page 18.

Two rammers, Figs. 112, 113 or 114, pages 373 and 374.

One mixing platform, about 15 feet square, built so substantially that it can be moved without coming to pieces, and having a 2 by 3-inch strip around the edge to prevent waste of materials and water. On a small job this may be of 1-inch stuff, resting on joists about 3 feet apart, provided it is stiffened by being tongued and grooved.

Fig. 3.—Square Pointed Shovel. (*See p.* 17.)

Fig. 4.—Concrete Wheelbarrow. (*See p.* 17.)

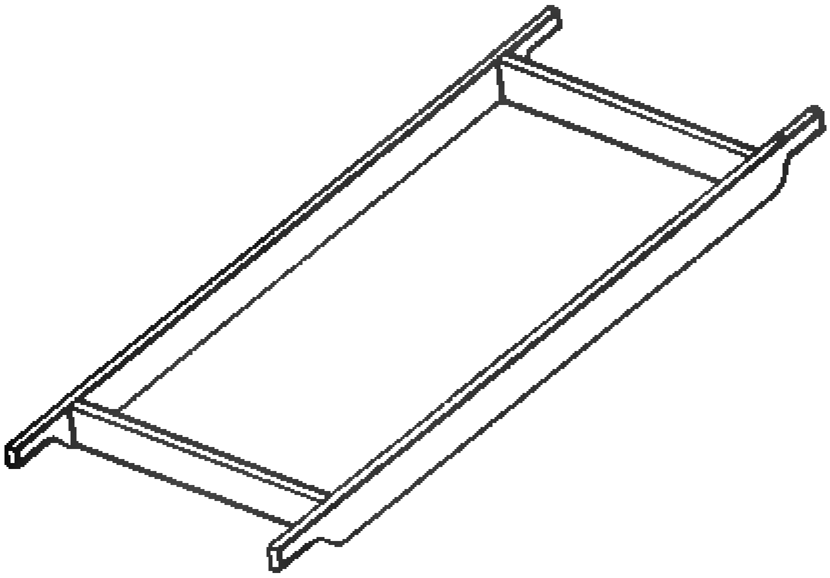

Fig. 5.—Measuring Box for Gravel. (*See p.* 18.)

One measuring box or barrel for sand, of a capacity for one batch of concrete. A convenient measure is a cement barrel, either whole or sawed in two, with both heads removed. It is filled and then lifted in such a manner as to spread the sand.

One measuring box for gravel (see Fig. 5) of a capacity for one batch of concrete.

Lumber for making and bracing forms.
Nails and, for some kinds of work, bolts, for forms.

CONSTRUCTION OF FORMS

Green spruce or fir lumber is suitable for forms. If a smooth face is required the surface of the boards or plank next to the concrete must be dressed and the edges tongued and grooved or beveled. The forms must be nearly water-tight. The sheeting, which is usually laid horizontal, may be 1 inch, 1½ inch or 2 inches thick, the distance apart of the studding being governed by the thickness selected. The studs must be placed not more than 2 feet apart for 1-inch sheeting nor more than 5 feet apart for 2-inch sheeting. They must be securely braced so as to withstand the pressure of the soft concrete and of the puddling or ramming.

The lower portion of a foundation wall in a trench excavated in earth so stiff as to stand nearly vertical may sometimes be laid with no form at all, and then narrowed in at the top to the required thickness, but if the sides of the trench are sloping it is generally cheaper to save concrete material by carrying the forms to the bottom. A thin wall may be

FIG. 6. — Construction of Form when Base of Wall is Spread. (*See p.* 19.)

greatly strengthened by spreading the base, which is readily accomplished by starting the boards or plank 6 or 8 inches above the bottom of the excavation and allowing the soft concrete to flow out under them on both sides of the wall so as to make footings, as shown in Fig. 6. The studs may run to the bottom, as indicated by the dotted lines, but should be tapered and greased so that they may be withdrawn without injury to the concrete.

For all walls under 9 or 10 inches in thickness, small steel rods ¼ or ⅜ inch in diameter, spaced about 18 inches apart, will greatly increase the stiffness and add to the safety of the structure, especially while the concrete is hardening.

Forms must be left in place for three or four weeks if there is earth or water pressure against the wall. If, on the other hand, there is no strain upon it, 24 hours setting, or until the concrete will stand the pressure of the thumb without indentation, is sufficient.

Further descriptions of form construction and methods of facing are given in Chapter XVII. Forms for special structures are described and illustrated in subsequent chapters treating of concrete design.

MIXING AND LAYING CONCRETE

The advisability of employing machinery for mixing the concrete depends chiefly upon the quantity to be laid. On a small job the first cost of mixing machinery and the running expenses, such as the labor of the engineman, which continue when the machine is idle, may bring the cost of machine-mixed concrete higher than hand-mixed. The decision may be based entirely upon the cost per cubic yard of concrete laid, provided a first-class machine is employed, since good concrete can be made either by machine or by hand. The various types of concrete mixers and the methods of employing them are discussed in Chapter XVI.

The foreman for a gang of concrete mixers need not be of great intelligence, but must be one who will obey orders strictly, and know how to keep all of his men constantly busy. The amount of work turned out will depend to quite an extent on the arrangement of the gang, whether each man has certain definite operations to perform over and over again, and whether these operations fit into the work of the rest of the gang so that none of the men have idle moments.

A gang of at least 6 men besides the foreman is required even on small work, while as many as 23 men may be effectively employed. In addition to these, an inspector is generally necessary to watch the placing of the

concrete and see that the mixture is uniform and of proper consistency. Cheap laborers, as for instance Italians, make good men for mixing and transporting the concrete.

The materials for the concrete ought, of course, to be deposited as near the work as possible. The cement, whether it comes in bags or barrels, must be sheltered from the rain. Covering with plank is insufficient. Bags should be protected from moist atmosphere; a cellar is likely to be too damp. To keep the sand and stone as near the mixing platform as possible, it may be advantageous to haul the materials as they are required from day to day. If the sand or stone pile is at any time farther from the measuring boxes than a man can profitably throw with shovels without walking, say more than 8 or 10 feet, do not hesitate to have it loaded into wheelbarrows and dumped into the measuring boxes. Materials can be wheeled in barrows to a distance of 10 to 25 feet from the platform at about the same cost that they can be shoveled direct with a long throw.

There are many methods of mixing concrete by hand, as discussed in Chapter XVI, all of which with care produce good work. For the convenience of the inexperienced the following directions for the work of a small gang of six men with foremen may be useful. They are given merely for illustration, and must be more or less varied to suit local circumstances.

Directions for Mixing Concrete. Assume a gang of four men to wheel and mix the concrete, with two other men to look after the placing and ramming.

When starting a batch, two mixers shovel or wheel sand into the measuring box or barrel — which should have no bottom or top — level it and lift off the measure, leveling the sand still further if necessary. They then empty the cement on top of the sand, level it to a layer of even thickness, and turn the dry sand and cement with shovels three times, as described below, after which the mixture should be of uniform color.

While these two men are mixing sand and cement, the other two fill the gravel measure about half full, then the two sand men take hold with them; and complete filling it. The gravel measure is lifted, the gravel hollowed out slightly in the center, and the mixture of sand and cement shoveled on top in a layer of nearly even thickness.* A definite number of pails are filled with water, and poured directly on the top of these layers, greater uniformity being thus attained than by adding the water directly from a hose. After soaking in slightly the mass is ready for turning.

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* Some engineers prefer to spread the stone on top of the sand and cement, while others prefer to mix the water with the sand and cement before adding them to the stone.

The method illustrated in Fig. 7 of turning with shovels materials which have already been spread in layers is as follows:

Two men, *a* and *b*, with square pointed shovels, stand facing each other at one end of the pile to be turned, one working right-handed and the other left-handed. Each man pushes his shovel along the platform under the pile, lifts the shovelful, turns with it, and then, turning the shovel completely over, and with a spreading motion drawing the shovel toward himself, deposits the material about 2 feet from its original position. Repetitions of this operation will form a flat ridge of the material, on a line with the pile as it originally lay, and flat enough so that the stones will not roll. As soon as, but not before, a single ridge is complete, two other men, *c* and *d*, should start upon this ridge, turning the materials for the second time, as shown in the illustration, and forming as before a flat ridge and finally a level pile which gradually replaces the last. A third mixing is accomplished in a similar way.

Fig. 7 gives the position of the piles as the concrete is being turned.

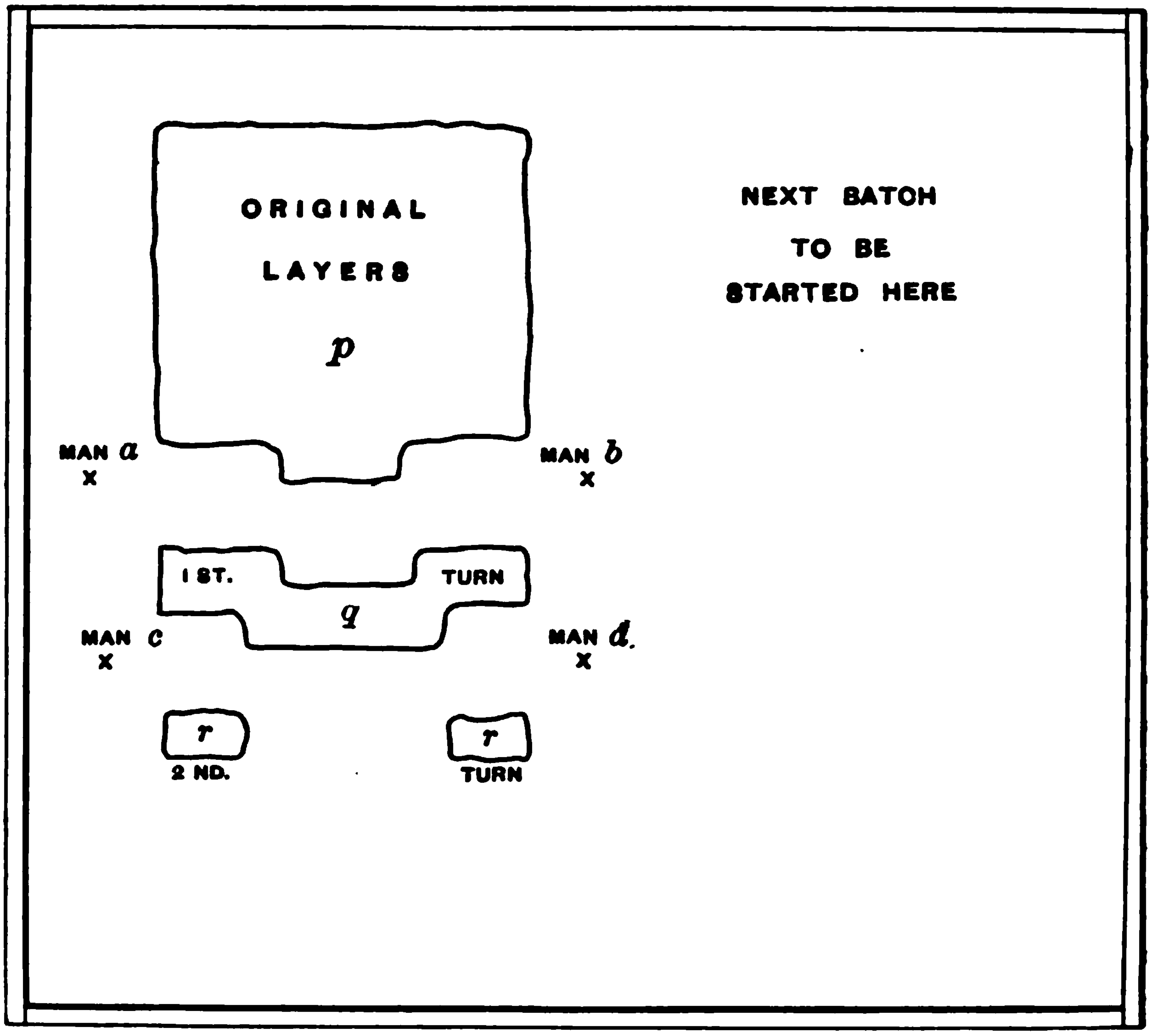

Fig. 7.—Position of Men and Concrete on Platform while Turning. (*See p.* 22.)

A portion of the original layers is shown at *p*, the ridge formed by men *a* and *b* shoveling from pile *p* is shown at *q*, and the beginning of the ridge formed by men *c* and *d* is shown at *rr*. The third turning is not shown.

The quantity of water used must be varied according to the moisture in the materials and the consistency required in the concrete. While the opinions of engineers regarding the proper consistency vary widely, it is advisable, the authors believe, for an inexperienced gang to use an excess of water. The rule may be made in hand mixing to use as much water as can be thoroughly incorporated with the materials. Concrete thus made will be so soft or "mushy" that it will fall off the shovel unless handled quickly.

After the material has been turned twice, as described, and as soon as the third turning has been commenced, two of the mixers who have finished turning may load the concrete into barrows and wheel to place. They should fill their own barrows, and after the mass has been completely turned for the third time by the other two men the latter should start filling the gravel measure for the next batch.

If the concrete is not wheeled over 50 feet, four experienced men ought to mix and wheel on the average about 10½ batches in ten hours. This figure is based on proportions 1: 2½: 5, and assumes that a batch consists of one barrel (four bags) Portland cement with 9.5 cubic feet of sand and 19 cubic feet of gravel or stone.

Assuming, as given on page 17, that 1.29 barrels of cement are required for 1 cubic yard of concrete, one barrel of cement — that is, one batch — will make 0.78 cubic yard of concrete; hence 10½ batches mixed and wheeled by four men in ten hours are equivalent to 8¼ cubic yards of concrete. This is for the very simplest kind of concreting and makes no allowance for the labor of supplying materials to the mixing platform or for building forms.

Placing Concrete. The concrete may be transported and handled by any means which will not cause the materials to separate. If mixed wet it may be dropped directly from shovels or barrows to place, or it may be run down an inclined pipe or chute. The layers should be about 6 inches thick. For a dry or a jelly-like mixture common square ended rammers are employed and the mass must be rammed until the mortar flushes to the surface. Wet concrete must be merely puddled or "joggled" to expel the air and surplus water. Before placing a fresh layer upon work which has set, the surface must be cleaned of dirt and scum, and thoroughly wet.

The placing of concrete and the kinds of rammers for different classes of work are discussed more at length in Chapter XVII.

APPROXIMATE COST OF CONCRETE

The cost of concrete depends more upon the character of the construction and the conditions which govern it than upon the first cost of the materials. In a very general way, we may say that when laid in large masses or in a very heavy wall, so that the construction of the forms is relatively a small item, the cost per cubic yard in place is likely to range from $4 to $7. The lower figure represents contract work under favorable conditions with low prices for materials, and the higher figure small jobs and inexperienced men. Similarly, we may say that for sewers and arches, where centering is required, the price may range from $7 to $14 per cubic yard. Thin building walls under eight inches thick may cost from $10 to $20 per cubic yard, according to the character of construction and the finish which is given to the surface.

These ranges in price seem enormous for a material which is ordinarily supposed to be handled by unskilled labor, but it must be borne in mind that skilled workmen are required for constructing forms and centers, and often the labor upon these may be several times that of mixing and placing the concrete. As a rule, unless the job is a very small one or under the personal supervision of a competent engineer, it is cheaper and more satisfactory to employ an experienced contractor than day labor. Green men under an inexperienced foreman may not be counted upon to mix and lay over one-half the amount of concrete that will be handled by a skilled gang under expert superintendence.

A close estimate of cost may be reached, in cases where the conditions are known in advance, by taking up in detail and then combining the various units of the material and labor as outlined below.

Cost of Cement. As the price of Portland cement varies largely with the demand, it is necessary to obtain quotations from dealers for every purchase. It is such heavy stuff that the freight usually enters largely into the cost, and quotations should therefore be made f.o.b. the nearest point of delivery to the work. The cost of hauling by wagon may be readily estimated by assuming that a barrel of cement weighs 400 pounds (gross), and that a pair of horses will haul over an average country road a load of, say, 5 000 pounds, traveling in all a distance of 20 to 25 miles in a day, that is, 10 to 12½ miles with load. This assumes, of course, that the teams are good and properly handled.

Having found the cost of the cement per barrel, delivered, the approximate cost per cubic yard is at once obtained from the table on page 17. If, for example, the cost is $2 per barrel and proportions 1 : 2½ : 5 are selected, the cost of the cement per cubic yard of concrete will be 1.29 × $2.00 = $2.58.

Cost of Sand. The cost of sand depends chiefly upon the distance hauled. With labor at 15 cents per hour, the cost of loading (including the cost of the cart waiting at pit) may be estimated, if handled in large quantities, at 18 cents per cubic yard, or on a small job at 27 cents per cubic yard. For hauling add one cent for each 100 feet of distance from the pit. The additional cost of screening, if required, will vary with the coarseness of the material, but 15 cents per cubic yard may be called an average price for this, unless the sand is obtained by screening the gravel, when no allowance need be made. After finding the cost of one cubic yard of sand, the cost of the sand per cubic yard of concrete is readily figured from the table referred to. If, for example, the cost of sand screened, loaded and hauled 1 000 feet is 52 cents per cubic yard, the cost per cubic yard of concrete for proportions 1 : 2½ : 5 will be 0.45 × $0.52 = $0.23½.

Cost of Gravel or Broken Stone. If broken stone is used upon a small job for the coarse aggregate, it is usually purchased by the ton or cubic yard. A 2000-lb. ton of broken stone may be considered as averaging approximately 0.9 cubic yards, although differences in specific gravity cause considerable variation. A two-horse load is generally considered 1½ to 2 yards, the latter quantity requiring very high sideboards. The cost of screening gravel, if this is necessary, while a very variable item, may be estimated at 35 cents per cubic yard. The cost of loading gravel into double carts, with labor at 15 cents per hour, may be estimated on a small job at 38 cents per cubic yard. If handled in large quantities, 25 cents is an average cost. The cost of loading includes loosening and also the cost of the cart waiting at the pit. Hauling costs about one cent per cubic yard additional for each 100 feet of distance hauled under load. If, to illustrate, the cost of gravel picked, screened, loaded and hauled 1 000 feet is 83 cents per cubic yard, the cost of the gravel per cubic yard of concrete for proportions 1 : 2½ : 5 will be 0.91 × $0.83 = $0.75½.

For distances up to 300 feet both sand and gravel can be hauled more economically by wheelbarrows than by teams. The cost of loading wheelbarrows is about half the cost of loading carts, while the cost of hauling with barrows per 100 feet is about four times greater.

Cost of Labor. With an experienced gang working at the rate of 15

cents per hour, the cost of mixing and laying concrete, if shoveled directly to place from the mixing platform, will average about 80 cents per cubic yard, in addition to the work on forms. If, as is usually the case, the concrete is wheeled in barrows, 9 cents per cubic yard must be added to the above price for the first 25 feet that the barrows are wheeled under load, and 1½ cents for each additional 25 feet wheeled. With other rates of wages, the cost may be considered as proportional. With a green gang, the cost will be nearly double the above figures, but as the men become worked in and the organization perfected, the cost should approximate more nearly the prices given.

The labor on forms is not included in the above. This is an extremely variable item. The cost of building rough plank forms (not including cost of lumber) on both sides of a 5-foot wall may be as low as 14 cents per cubic yard of concrete, with other thicknesses of wall in inverse proportion. On elaborate work the price, which is really dependent upon the face area, may reach several dollars per cubic yard of concrete.

THE STRENGTH OF CONCRETE

The strength of concrete varies (1) with the quality of the materials; (2) with the quantity of cement contained in a cubic yard of the concrete; and (3) with the density of the mixture.

We may say that the strongest and most economical mixture consists of an aggregate comprising a large variety of sizes of particles, so graded that they fit into each other with the smallest possible volume of spaces or voids, and enough cement to slightly more than fill all of these spaces or voids between the solids of the aggregate. It is obvious that with the same aggregate the strongest cement will make the strongest concrete.

On important construction the various materials to be used should be carefully tested, and specimens of the mixture selected made up in advance and subjected to test. As a guide to the loads which concrete will stand in compression, — that is, under vertical loading where the height of the column or mass is not over, say, 12 times the least horizontal dimension, — we may give the following approximate figures as safe strengths, after the concrete has set at least one month, for the proportions which have previously been selected in this article as typical mixtures.

The figures, compared with the results of recent experiments on 12-inch cubes, allow a factor of safety of six at the age of one month, or eight at the age of six months, and are based on conservative practice. The relative

strengths of the different mixtures are calculated from original investigations of the authors discussed in Chapter XIII.

Safe Strength of Portland Cement Concrete in Direct Compression.

Proportions.	Pounds per square inch.	Tons per square foot.
1:2:4	410	29
1:2½:5	360	25
1:3:6	325	23
1:4:8	260	18

With a large mass foundation take values one-eighth greater.
With a vibrating or pounding load, take one-half these values.

The tensile strength of concrete is very much less than the compressive strength. Experiments made by the authors, with mixtures of average proportions, give the ultimate fiber stress in beams as about one-eighth the breaking strength in compression.

CHAPTER III

SPECIFICATIONS

In the following pages are given specifications for

Cement, in brief, for the small purchaser. (See p. 29.)
Portland cement, in full, for the large purchaser. (See p. 29.)
Natural cement, in full, for the large purchaser. (See p. 31.)
Portland cement concrete. (See p. 32.)
First-class steel for reinforced concrete. (See p. 38.)

These specifications cover all ordinary concrete construction, and are adapted as far as possible for direct use in placing contracts for material and construction, although concrete specifications for structures of intricate design will require the insertion of additional paragraphs referring specifically to the particular work.

If sand, screenings, gravel, stone, or steel are purchased on separate contracts, paragraphs 2, 3, 4, 5, or 7 (pp. 33 and 34) may be extracted from the concrete specifications.

The full specifications for cement are advised for important work, whether large or small, although the brief specifications which precede them may be sometimes useful.

Even when purchasing by the full specifications it may often be, and in fact in the majority of cases it is, unnecessary actually to test the cement, except for soundness and fineness, but the strict detail specifications are necessary so that if the cement is found to work unsatisfactorily samples may be subjected to complete tests on the ground, or sent to testing laboratories, and the remainder of the shipment or subsequent shipments rejected.

Printed specifications are frequently preceded by a "Notice to Contractors" stating the place and time of receiving bids, the amount of the check to be deposited with each bid and the bond required, and specifying that the contractor shall give references and shall state what work of a similar character he has performed. A "Form of Bid" is also sometimes inserted.

The specifications and contract are based upon the authors' practice supplemented by a careful study of specifications of the American Society for Testing Materials, of the American Railway Engineering & Maintenance-of-Way Association, of the City of Philadelphia, of the United States

Army, of the United States Navy, of the Massachusetts Metropolitan Commissions, of the New York Rapid Transit Commission, and others.

BRIEF SPECIFICATIONS FOR PURCHASE OF CEMENT

The cement shall be a first-class Portland† cement of a standard brand bearing a good reputation, sound, *i.e.*, not liable to expansion or disintegration, — fine and of uniform quality. It shall be free from lumps and shall be packed in sound barrels.‡

FULL SPECIFICATIONS FOR PURCHASE OF PORTLAND CEMENT

1. **Packages.** Cement shall be packed in strong cloth or canvas sacks.§ Each package shall have printed upon it the brand and name of the manufacturer. Packages received in broken or damaged condition may be rejected or accepted as fractional packages.

2. **Weight.** Four bags shall constitute a barrel, and the average net weight of the cement contained in one bag shall be not less than 94 lb. or 376 lb. net per barrel. A cement bag may be assumed to weigh one pound. The weights of the separate packages shall be uniform.

3. **Requirements.*** Cement failing to meet the seven-day requirements may be held awaiting the results of the twenty-eight-day tests before rejection.

4. **Tests.*** All tests shall be made in accordance with the methods proposed by the Committee on Uniform Tests of Cement of the American Society of Civil Engineers, presented to the Society January 21, 1903, and amended January 20, 1904, with all subsequent amendments thereto. (See Chapter VII, page 63.)

5. **Sampling.** Samples shall be taken at random from sound packages, and the cement from each package shall be tested separately.

6.* The acceptance or rejection shall be based on the following requirements:

7. **Definition of Portland Cement.*** This term is applied to the finely pulverized product resulting from the calcination to incipient fusion of an intimate mixture of properly proportioned argillaceous‖ and calcareous¶ materials, and to which no addition greater than 3% has been made subsequent to calcination.

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*Paragraphs designated by an asterisk are quoted from the Standard Specifications of the American Society for Testing Materials.

†Or Natural, or Puzzolan.

‡If stored in a dry place to be used immediately, it may be packed in stout cloth or canvas bags which are of course cheaper than barrels.

§If the cement is to be stored in a damp place or near the sea, it must be packed in well-made wooden barrels lined with paper.

‖Clayey.

¶Consisting chiefly of lime or calcium.

8. **Specific Gravity.*** The specific gravity of the cement, thoroughly dried at 100° Cent. (212° Fahr.), shall be not less than 3.10.

9. **Fineness.*** It shall leave by weight a residue of not more than 8% on the No. 100, and not more than 25% on the No. 200 sieve.

10. **Time of Setting.*** It shall develop initial set in not less than thirty minutes, but must develop hard set in not less than one hour nor more than ten hours.

11. **Tensile Strength.**† Briquettes one inch square in section shall attain at least the following tensile strengths and shall show no retrogression within the periods specified.

Neat Cement.

Age	Strength†
24 hours in moist air	175 lb.
7 days (1 day in air, 6 days in water)	500 "
28 days (1 " " 27 " ")	600 "

One Part Cement, Three Parts Standard Sand.

Age	Strength†
7 days (1 day in moist air, 6 days in water)	150 lb.
28 days (1 " " " 27 " ")	200 "

12. **Soundness or Constancy of Volume.*** Pats of neat cement about three inches in diameter, one-half inch thick at the center, and tapering to a thin edge, shall be kept in moist air for a period of twenty-four hours.

(*a*) A pat is then kept in air at normal temperature, and observed at intervals for at least 28 days.

(*b*) Another pat is kept in water maintained as near 70° Fahr. as practicable, and observed at intervals for at least 28 days.

(*c*) A third pat is exposed in any convenient way in an atmosphere of steam, above boiling water, in a loosely closed vessel for five hours.

These pats to satisfactorily pass the requirements shall remain firm and hard and show no signs of distortion, checking, cracking or disintegration.

13. **Sulphuric Acid and Magnesia.** The cement shall not contain more than 1.75% of anhydrous sulphuric acid (SO_3), nor more than 4% of Magnesia (MgO).

*Paragraphs designated by an asterisk are quoted from the Standard Specifications of the American Society for Testing Materials.

†The American Society for Testing Materials gives minimum requirements as follows: Neat Cement — 24 hours, 150–200 lb., 7 days, 450–550 lb., 28 days, 550–650 lb. 1:3 mortar — 7 days, 150–200 lb., 28 days, 200–300 lb.; the exact values to be fixed in each case by the consumer.

FULL SPECIFICATIONS FOR THE PURCHASE OF NATURAL CEMENT

1. **Packages.** Cement shall be packed in strong cloth or canvas sacks.† Each package shall have printed upon it the brand or the name of the manufacturer. Packages received in broken or damaged condition may be rejected or accepted as fractional packages.

2. **Weight.** Three bags shall constitute a barrel, and the average net weight of the cement contained in one bag shall be not less than 94 lb., or 282 lb. net per barrel. A cement bag may be assumed to weigh one pound. The weights of the separate packages shall be uniform.

3. **Requirements.*** Cement failing to meet the seven-day requirements may be held awaiting the results of the twenty-eight day tests before rejection.

4. **Tests.*** All tests shall be made in accordance with the methods proposed by the Committee on Uniform Tests of Cement of the American Society of Civil Engineers, presented to the Society January 21, 1903, and amended January 20, 1904, with all subsequent amendments thereto. (See Chapter VII, p. 63.)

5. **Sampling.** Samples shall be taken at random from sound packages, and the cement from each package shall be tested separately.

6.* The acceptance or rejection shall be based on the following requirements:

7. **Definition of Natural Cement.*** This term shall be applied to the finely pulverized product resulting from the calcination of an argillaceous limestone at a temperature only sufficient to drive off the carbonic acid gas.

8. **Specific Gravity.*** The specific gravity of the cement thoroughly dried at 100° Cent. (212° Fahr.) shall be not less than 2.8.

9. **Fineness.*** It shall leave by weight a residue of not more than 10% on the No. 100, and 30% on the No. 200 sieve.

10. **Time of Setting.*** It shall develop initial set in not less than ten minutes, and hard set in not less than thirty minutes, nor more than three hours.

11. **Tensile Strength.** Briquettes one inch square in section shall attain

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*Paragraphs designated by an asterisk are quoted from the Standard Specifications of the American Society for Testing Materials.

†If the cement is to be stored in a damp place or near the sea, it must be packed in well-made wooden barrels lined with paper.

at least the following tensile strength and shall show no retrogression within the periods specified:

Neat Cement.

Age	Strength†
24 hours in moist air	50 lb.
7 days (1 day in air, 6 days in water)	100 "
28 days (1 " " 27 " ")	200 "

One Part Cement, Three Parts Standard Sand.

Age	Strength†
7 days (1 day in air, 6 days in water)	25 lb.
28 days (1 " " 27 " ")	75 "

12. **Constancy of Volume.*** Pats of neat cement about 3 inches in diameter, one-half inch thick at the center, and tapering to a thin edge, shall be kept in moist air for a period of 24 hours.

(*a*) A pat is then kept in air at normal temperature.

(*b*) Another pat is kept in water maintained as near 70° Fahr. as practicable.

These pats are observed at intervals for at least 28 days, and to satisfactorily pass the tests should remain firm and hard and show no signs of distortion, checking, cracking, or disintegrating.

CONTRACT AND SPECIFICATIONS FOR PORTLAND CEMENT CONCRETE‡

This agreement made this .... day of .... in the year 19....

by and between (Name of party letting the contract.) of ....,

party of the first part, and (Name of accepted contractor.) of ....,

party of the second part.

Witnesseth: That the parties to these presents, each in consideration of the covenants and agreements on the part of the other, herein contained, have covenanted and agreed, and do hereby covenant and agree, for themselves and their heirs, executors, administrators, and assigns, and under the

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*Paragraphs designated by an asterisk are quoted from the Standard Specifications of the American Society for Testing Materials.

†The American Society for Testing Materials gives minimum requirements as follows: Neat Cement — 24 hours, 50–100 lb., 7 days, 100–200 lb., 28 days, 200–300 lb. 1:3 mortar — 7 days, 25–75 lb., 28 days, 75–150 lb., the exact values to be fixed in each case by the consumer.

‡For Natural cement concrete paragraphs 1, 11 and 14 must be slightly altered, and paragraphs 7 and 13c omitted.

penalty expressed in a bond bearing even date with these presents, and hereto annexed, as follows:

The contractor shall begin work within .... days of the date of this contract, and shall, at his own proper cost and expense, provide and deliver all of the materials and perform all of the work called for by these specifications, and supply all implements, apparatus, and appliances needed in performing the work.

The entire work shall be completed on or before ....

19...........[*]

1. **Cement.**[†] The cement shall be first-class Portland cement of reputable brand which shall conform in all respects to the cement specifications herewith annexed. The cement shall be stored in a building which will protect it from the weather. The floor upon which the cement is placed shall be at least 6 inches above the ground. It shall be stored so as to permit of easy access for inspection and indentification of each shipment. A sufficient quantity shall be kept on hand at all times so that the Engineer may have opportunity and time to make tests sufficient to determine its quality. At least 12 days shall be allowed for inspection and necessary tests.

2. **Sand.**[‡] The sand shall be clean and coarse, or a mixture of coarse and fine grains with the coarse grains predominating.[§] It shall be free from clay, loam, sticks, organic matter, and other impurities.

3. **Screenings.**[‖] Screenings or crusher dust from broken stone, — in which term is included all particles passing a ¼-inch screen, — may, by slightly altering the proportions of the ingredients, be substituted for the whole or a portion of the sand in such proportions as to give a dense mixture and the same relative volumes of total aggregate.

4. **Gravel.**[‖] The gravel shall be composed of clean pebbles free from

[*] A premium and forfeiture clause may here be inserted, but a forfeiture clause without a premium in many cases cannot be legally enforced. The word "penalty" should never be employed.

[†] It is often advisable that the cement be furnished by the party letting the contract or, to prevent waste of cement, that it be sold by them to the contractor at an arbitrary price per barrel, — say, about one-half the actual cost of the cement, — which price must be definitely stated in the contract.

[‡] Concrete construction is not prohibited if sand of the quality designated is unobtainable, but coarse sand should usually be selected in preference to fine, even if its cost is double or three times the latter. The quality of sharpness is purposely omitted. (See p. 153.)

[§] A definite percentage above a certain diameter may be required. (See p. 149.)

[‖] Omit paragraphs for materials which are not used. If two or more sizes of any aggregate are used, define them.

sticks and other foreign matter and containing no clay or other material adhering to the pebbles in such quantity that it cannot be lightly brushed off with the hand or removed by dipping in water. It shall be screened[*] to remove the sand, which shall afterwards be remixed with it in the required proportions.

5. **Broken Stone.**[†] The broken or crushed stone shall consist of pieces of hard and durable rock, such as trap, limestone, granite, or conglomerate. The dust shall be removed by a ¼-inch screen, to be afterwards, if desired, mixed with and used as a part of the sand, except that if the product of the crusher is delivered to the mixer so regularly that the amount of dust, as determined by frequently screening samples, is uniform, the screening may be omitted and the average percentage of dust allowed for in measuring the sand.

6. **Water.** The water shall be free from acids or strong alkalies.

7. **Reinforcing Steel**[†][‡]. Steel for reinforcement shall have an "ultimate tensile strength of 55,000 to 65,000 pounds per square inch, an elastic limit of not less than one-half the ultimate strength (*i. e.* not less than 27,000 lb.) and a minimum elongation in 8 inches of 1 400 000 divided by the ultimate strength per cent."[§] Metal reinforcement shall be of such shape or so anchored as suitably to assist its adhesion to the concrete.

8. **Proportions.** The proportions of the raw materials for the concrete shall be exactly determined from time to time by the Engineer in accordance with the relative coarseness of the aggregate, so as to attain as dense a concrete as is consistent with the terms of the specifications which follow. The following paragraphs designate the average relative volumes of material for each class of work.

For ....[||], one barrel (376 lb.) cement to .... cubic feet sand[¶] to .... cubic feet broken stone,[¶] the cement to be measured as packed by the manufacturer, and the sand and other aggregate to be measured as shoveled loosely into an ordinary sand or stone measuring box or barrel.

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[*] In exceptional cases where the relation of pebbles to sand is very uniform, the mixture of sand and pebbles may be used without screening. Frequent tests shall then be made to see that the proportions of the coarse and fine grains are correct.

[†] Omit paragraphs for materials which are not used. If two or more sizes of any aggregate are used, define them.

[‡] Specifications for high carbon steel are given in full on page 38. High carbon steel is distrusted by many, but may be safely employed if it fulfils the requirements there given, and owing to its greater strength will be more economical than ordinary merchant steel.

[§] Suggested for structural steel by the Committee on Boston Building Laws of the Boston Society of Civil Engineers.

[||] Insert a description of portion of structure. Repeat paragraph as required.

[¶] If other materials are selected for the aggregate alter the wording accordingly.

9. **Hand Mixing.** If the concrete is mixed by hand, the cement and aggregate shall be mixed and the water added on a tight platform large enough to provide space for the partially simultaneous mixing of two batches of not more than one cubic yard each. The sand and cement shall be spread in thin layers and mixed dry until of uniform color. This mixture may be spread upon the layer of stone or the stone shoveled upon it before adding the water, or it may be made into a mortar before spreading it with the stone. In the former method the materials shall be turned at least three times, — in addition to the mixing of the sand and cement already mentioned, the water being added on the first turning, — and in addition to the shoveling from the platform to place or into the vehicle for transportation. In the latter method, that is, if the sand and cement are first made into a mortar, the mass of mortar and stone shall be turned at least twice. Whatever method is employed, the number of turnings shall be sufficient to produce a resulting loose concrete of uniform color and appearance, with the stones thoroughly incorporated into the mortar and the consistency uniform throughout.

10. **Machine Mixing.** If the concrete is mixed in a machine mixer a machine shall be selected into which the materials, including the water, can be precisely and regularly proportioned, and which will produce a concrete of uniform consistency and color with the stones and water thoroughly mixed and incorporated with the mortar.

11. **Consistency.** (*a*) A medium or quaking mixture of a tenacious, jelly-like consistency, which quakes on ramming, shall be used for ordinary mass concrete, such as foundations, heavy walls, large arches, piers, and abutments.

(*b*) Very wet or mushy concrete, so soft that it must be handled quickly or it will run off the shovel, shall be used for rubble concrete, and for reinforced concrete, such as thin building walls, columns, doors, conduits, and tanks.

(*c*) Dry concrete, of the consistency of damp earth, may be employed in dry locations for mass foundations, which must withstand severe compressive strain within one month after placing, provided it is spread in 6-inch layers and rammed until water flushes to the surface. Dry mixed concrete shall never be employed with steel reinforcement.

12. **Placing.** Concrete shall be conveyed to place in such a manner that there shall be no distinct separation of the different ingredients, or, in cases where such separation inadvertently occurs, the concrete shall be remixed before placing. Each layer in which the concrete is placed shall be of such

thickness that it can be incorporated with the one previously laid. Concrete shall be used so soon after mixing that it can be rammed or puddled in place as a plastic homogeneous mass. Any which has set before placing shall be rejected. When placing fresh concrete upon an old concrete surface, the latter shall be cleaned of all dirt and scum or laitance, and thoroughly wet. Noticeable voids or stone pockets discovered when the forms are removed shall be immediately filled with mortar mixed in the same proportions as the mortar in the concrete. *(For horizontal joints in thin walls, or in walls to sustain water pressure† or in other important locations, a joint of mortar in proportions designated by the Engineer may be required, and no allowance over and above the normal unit price shall be made to the contractor for the material or labor used.)

13*a*.‡ **Ordinary Surface.** Surfaces shall have no special treatment further than care in placing the concrete to avoid noticeable voids or stone pockets. Forms shall be wet (except in freezing weather) before placing the concrete against them.

13*b*.‡ **Exposed Faces.** Faces exposed to view shall be made smooth by thrusting a spade or chisel through the concrete close to the form to force back the large stones and prevent stone "pockets." The forms shall be greased with crude oil before placing the concrete against them. On removal of the forms, surfaces shall be ....§

13*c*.‡ **Mortar Surface.** Moldings, cornices, and other ornaments requiring mortar surface, shall be formed by spreading plastic mortar upon the interior of finely constructed molds, just as the concrete is being laid. No exterior plastering shall be permitted.

14. **Freezing Weather.**¶ No concrete, except that laid in large masses, or heavy walls having faces whose appearance is of no consequence, shall be exposed to frost until hard and dry. Materials employed in mass concrete in freezing weather shall contain no frost. Surfaces shall be protected from frost. Portions of surface concrete which have frozen shall be removed before laying fresh concrete upon them.

15. **Forms.** The lumber for the forms and the design of the forms shall be adapted to the structure and to the kind of surface required on the concrete. For exposed faces the surface next to the concrete shall be dressed. Forms shall be sufficiently tight to prevent loss of cement or

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*Omit following sentence in ( ) unless the work includes this class of structure.

†Tanks and other structures having thin walls to resist water pressure should be built as monoliths, that is, with no interruption in the work, proceeding, if necessary, night and day.

‡Select one or more paragraphs from 13*a*, 13*b*, and 13*c*.

§State kind of finish desired. (See p. 380.)

¶Natural cement concrete must *never* be exposed to frost until thoroughly hard and dry.

mortar. They shall be thoroughly braced or tied together so that the pressure of the concrete, or the movement of men, machinery or materials, shall not throw them out of place. Forms shall be left in place until, in the judgment of the Engineer, the concrete has attained sufficient strength to resist accidental thrusts and permanent strains which may come upon it. Forms shall be thoroughly cleaned before being used again.

16. **General Requirements.** Imperfect work or materials, or work or materials which may become damaged from any cause before its acceptance, shall be properly replaced to the satisfaction of the Engineer.

Foremen employed by the contractor shall be skilled in concrete mixing, and they shall receive and obey orders from the Engineer.

No claims for extra work shall be allowed unless made in writing previous to its performance and signed by both parties or by their authorized representatives.

In case of disagreement as to the meaning of the terms of the contracts or as to the manner of its execution, one arbitrator shall be appointed by each party within one week after notification in writing by either party, and in case these cannot agree, a third arbitrator shall be selected by these two, and the decision of the majority of the arbitrators shall be final and binding on both parties.

17. **Prices for Work.** The following prices shall be paid to the contractor as full compensation for the furnishing and use of all materials and implements required on the work and for all labor.

(Here shall be inserted all unit prices for all divisions of the work, or the lump sum for the entire work, or the lump sums for different divisions of the work, or for alternate proposals, followed by a paragraph stating the manner and time of payments and the amount withheld each month.)

In witness whereof the parties to these presents have affixed their hands

and seals this .... day of ...., 19...........

Signed in the presence of

....(Seal)

....

....(Seal)

....

Bond to Accompany the Contract.*

Know all men by these presents, That we

....

as sureties, are held and firmly bound unto..
in the sum of..dollars
($.....................), to be paid said.., for which
payment, well and truly to be made, we bind ourselves, our heirs, executors and administrators, jointly and severally, firmly by these presents.

The condition of this obligation is such, that if the above bounden

....

heirs, executors, administrators or assigns, shall in all things stand to and abide by, and well and truly keep and perform, the covenants, conditions and agreements in the foregoing contract on his or their part to be kept and performed, at the time and in the manner therein specified, and shall indemnify and save harmless the said..
as therein stipulated, then this obligation shall become and be null and void; otherwise it shall be and remain in full force and virtue.

In witness whereof we hereunto set our hands and seals on this................
................................day of..in the year nineteen hundred and ....(Seal)

....(Seal)

Signed and sealed in presence of ....

....

SPECIFICATIONS FOR FIRST-CLASS STEEL TO BE USED IN REINFORCED CONCRETE†

1. **Process of Manufacture.** Steel shall be made by the open hearth process.

2. **Chemical Properties.** Steel shall conform to the following limits in chemical composition:

Phosphorus shall not exceed 0.06.
Sulphur shall not exceed 0.06.
Manganese shall not exceed 0.80 or be below 0.40.

3. **Physical Properties.** The steel shall conform to the following physical qualities:

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*Form adopted by Metropolitan Commissioners, Mass.

†Steel of this hardness should not be used unless enough of it is to be bought to warrant the making of complete tests as per specifications. Ordinary mild steel may be purchased in the open market without specifications. In using steel bought in open market, it is not safe to count on a tensile strength greater than 55 000 lb. — Frederick W. Taylor.

4. *Tensile Tests.* Tensile strength in pounds per square inch shall be not less than 105 000

Yield point in pounds per square inch shall be not less than 52 500

Elongation per cent. in eight inches shall be not less than 10

5. For material less than five sixteenths inch (5/16") and more than three fourths inch (3/4") in thickness the following modifications shall be made in the requirements for elongation:

(*a*) For each increase of one eighth inch (1/8") in thickness above three fourths inch (3/4") a deduction of one per cent. (1%) shall be made from the specified elongation.

(*b*) For material from 1/4 inch to, but not including, 5/16 inch thick the elongation shall be 8%.

For material from 3/16 inch to, but not including, 1/4 inch thick the elongation shall be 7%.

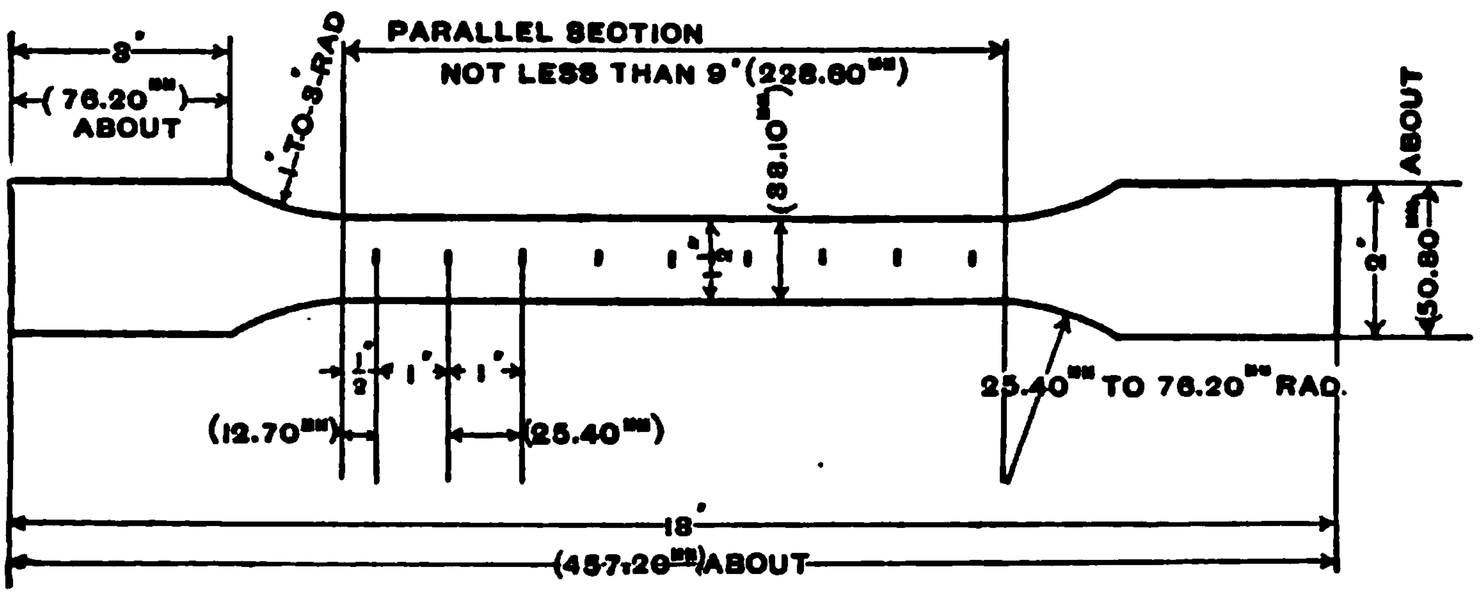
For material from 1/8 inch to, but not including, 3/16 inch thick the elongation shall be 6%.

For material less than 1/8 inch thick the elongation shall be 5%.

6. *Bending Test.* Test specimens for bending* shall be bent cold around a diameter equal to their thickness to the following angles without fracture on the outside of the bent portion:

For specimens 1 inch thick 80°. For specimens 3/4 inch thick 90°.
For specimens 1/2 inch thick 110°. For specimens 1/4 inch thick 130°.
For specimens 3/16 inch thick 140°. For specimens 1/8 inch thick 160°.

7. **Test Pieces and Methods of Testing.** Where practicable the standard test specimen of eight-inch (8") gaged length shall be used to determine the physical properties specified in paragraphs Numbers 4 and 5. The standard shape of the test specimen for sheared plates shall be as shown by the following sketch:



.PIECE TO BE OF SAME THICKNESS AS PLATE

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*The most important test of all is the *bending* test, but any soft steel will stand the bending test, so that the tensile test is needed to secure a steel which is strong enough.

For material from which it is impracticable to obtain test specimens like those for sheared plates, the test specimen may be planed or turned parallel throughout its entire length, and in all cases where possible two opposite sides of the test specimen shall be the rolled surfaces. Small rolled bars of uniform section shall be tested full size as rolled.

8. All test specimens shall be cut from the finished material as it comes from the rolls, unless such materials are to be annealed, in which case the test specimens will be taken after the annealing process. In case several shapes are rolled from one heat, two test specimens will be taken from two different shapes, representing their class, for tension, and two for bending. When only one shape is rolled from a heat, two test specimens for tension and two for bending will be taken from each ten tons or fraction thereof.

9. Where practicable the bending test specimen shall be one and one-half inches (1½″) wide, and for material three-quarters inch (¾″) and less in thickness, this specimen shall have the natural rolled surface on two opposite sides. For material more than three-quarters inch (¾″) thick, the bending test specimen may be cut to one-half inch (½″) thick.

10. The bending test may be made by pressure or by blows.

11. In case a test specimen develops flaws or in case it breaks outside of the middle third of its gaged length, it may be discarded and another test specimen substituted therefor.

12. For the purposes of this specification, the yield point shall be determined by the careful observation of the drop of the beam, or halt in the gage, of the testing machine.

13. In order to determine if the material conforms to the chemical limitations prescribed in paragraph No. 2 herein, analysis shall be made of clean drillings taken from a small test ingot.

14. **Variation in Weight.** A variation in cross section or weight of more than 2½ per cent. from that specified will be sufficient cause for rejection.

15. **Finish.** Finished material must be free from injurious seams, flaws, or cracks, and have a workmanlike finish.

16. **Annealing.** All bars which, owing to their shape or size, are liable to be under strain after cooling, must be reheated to a temperature not less than 1250° (Fahrenheit) nor more than 1375°, and this heating and subsequent cooling must be done in an approved manner.

CHAPTER IV

THE CHOICE OF CEMENT

When the construction under consideration is not of a grade to warrant the testing of different cements before making a selection, the question often arises as to whether, for example, Portland or Natural cement is most desirable from the standpoint of economy, or whether common lime or a mixture of lime and cement is suitable for the purpose.

Although the decision must often depend upon local conditions, a few general rules may be formulated relating to the classes of construction for which different kinds of cement and lime are adapted, followed by illustrations of the methods for making a selection where there is a choice between two cements and between different brands of the same cement.

THE CLASS OF CEMENT

Portland Cement should be used in concrete and mortar for structures subjected to severe or repeated stresses; for structures requiring strength at short periods of time; for concrete building construction; for work laid under water or with which water will come in contact immediately after placing; for thin walls subjected to water pressure; for masonry exposed to wear or to the elements; and for all other purposes where its cost will be less than that of Natural cement concrete, or mortar of similar quality.

Natural Cement may be substituted for Portland in concrete, if economy demands it, for dry unexposed foundations where the load in compression can never exceed, say, 75 lb. per square inch (5 tons per sq. ft.) and will not be imposed until three months after placing; for backing or filling in massive concrete or stone masonry where weight and mass are the essential elements; for sub-pavements of streets, and for sewer foundations.

In mortar Natural cement is adapted for ordinary brickwork not subjected to high water pressure or to contact with water until, say, one month after laying, and for ordinary stone masonry where the chief requisite is weight and mass.

Natural cement concrete or mortar should never be allowed to freeze, should never be laid under water, in exposed situations, in columns, beams, floors or building walls, or in marine construction.

Mixtures of Portland and Natural Cements, unless mixed at the factory and sold as Improved Natural Hydraulic Cements, are not advised under any conditions.

Sand Cement* is recommended by the United States Army Engineers for grouting†, and it is sometimes employed as a substitute for Natural cement. Its use in place of pure Portland cement is often worth investigation and testing in combination with the aggregate.

Puzzolan or Slag Cements are limited to certain proper uses by the engineer officers of the U. S. Army‡ as follows:

> Puzzolan cement never becomes extremely hard like Portland, but Puzzolan mortars and concretes are tougher or less brittle than Portland.
>
> The cement is well adapted for use in sea water,§ and generally in all positions where constantly exposed to moisture, such as in foundations of buildings, sewers, and drains, and underground works generally, and in the interior of heavy masses of masonry or concrete.
>
> It is unfit for use when subjected to mechanical wear, attrition, or blows. It should never be used where it may be exposed for long periods to dry air, even after it has well set. It will turn white and disintegrate, due to the oxidation of its sulphides at the surface under such exposure.

Hydraulic Lime, which has the property of setting under water, is extensively employed on the continent of Europe, especially in France, when in the United States common lime would be used, and frequently in place of hydraulic cement. Beton-Coignet is a mixture of hydraulic lime with cement and sand. Candlot‖ gives as the proportions most frequently employed, 1 cubic meter (35.3 cu. ft.) sand, 125 to 150 kilograms (276 to 331 lb.) lime, and 50 to 60 kilograms (110 to 132 lb.) cement. Hydraulic lime is not manufactured in the United States.

Common Lime is not suitable for a principal ingredient in concrete. It will not set in contact with water, sustain heavy loads, or resist wear.

The use of lime mortar, in the building laws of some cities, is limited to chimney construction in frame buildings, while other cities permit its use in walls of all except fireproof buildings. The Boston building laws (1896) limit the stresses on brick laid in lime mortar to 7 tons per square feet.

Lime and Natural Cement mortar is suitable for ordinary building brickwork, for light rubble foundations and for building walls.

Lime and Portland Cement mortar is adapted for the same purposes

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*See page 48.

†Professional Papers No. 28.

‡Professional Papers No. 28.

§See Chapter XVIII, by R. Feret.

‖Ciments et Chaux Hydrauliques, 1898, p. 289.

as mortars of lime and Natural cement, but are of superior quality and strength.

Hydrated Lime* is preferable to common lime paste or putty for use with Portland cement, because if properly manufactured it is more thoroughly slaked and is easily handled and measured.

Choice Determined by Cost. — When the character of the structure admits of either Portland or Natural cement, the choice is based upon the relative cost, which, in turn, is dependent upon the proportions that may be adopted in either case. The sand in Portland cement mortar is usually limited, by practical considerations of handling with the trowel, to proportions 1:3 in some instances and to 1:4 in others, while the most common proportions for Natural cement mortar are 1:2, that is, one part cement to two parts sand, by volume.

The relative cost, after assuming the proportions of the two substitute classes of mortar, is governed primarily by the quantity of cement in a cubic yard of mortar. For example, from table on page 229, 3.32 bbl. of cement (based on a barrel of 3.8 cu. ft.) are required per cubic yard of 1:2 mortar, while 2.48 bbl. are required for 1:3 mortar. Hence, if a decision lies between 1:2 Natural mortar and 1:3 Portland mortar, and the smaller item of quantity of sand is disregarded, the mortar produced from Natural cement at $1.00 per barrel will cost the same as that produced from Portland cement at $(\$1.00 \times \frac{3.32}{2.48}) = \$1.34$ per barrel. Similarly, since 1:4 mortar requires 1.98 bbls. of cement per cubic yard, Portland cement mortar one part cement to 4 parts sand is equivalent in cost to 1:2 Natural cement mortar when Natural cement is $1.00 per barrel and Portland cement is $(\$1.00 \times \frac{3.32}{1.98}) = \$1.68$ per barrel; that is, when Portland cement delivered on the job costs 68% more than Natural cement. Allowance for difference in quantity of sand brings the Portland values still lower, as shown in the table on page 45. With Portland and Natural cement mortars of equal cost, the Natural cement produces brickwork of lower cost because, a fact usually overlooked in estimates, a bricklayer can lay in a given time about 10% more brick with Natural cement mortar of proportions 1:2 than with Portland cement mortar of proportions, say, 1:3.

From the results of the comparatively few available tests, Portland cement concrete at the age of six months appears to be at least three times

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*See S. Y. Brigham in *Engineering News*, Aug. 27, 1903, p. 177, and Charles Warner in *Rock Products*, Feb., 1904, p. 26.

as strong as Natural cement concrete in the same proportions, while at earlier periods the ratio is still larger. Since Portland cement concrete mixed 1: 2: 4 is only about 1½ times stronger than a 1:4: 8 Portland mixture, it is very evident that the choice between Portland and Natural cement for concrete is determined, as in mortars, by practical considerations other than relative strength.

The following elementary example illustrates the method of estimating the comparative cost of Portland and Natural cement concrete:

Example. — What price can be paid per barrel for Portland cement to make a concrete 1: 4: 8 of equivalent cost to a 1: 2: 4 Natural cement concrete, if Natural cement costs $1.00 per barrel, sand $0.75 per cubic yard, and stone having 45% voids $1.50 per cubic yard?

Solution. — By reference to the table of quantities of materials on page 17, we find that the 1: 2: 4 Natural concrete will cost per cubic yard for materials only:

1.57 barrels cement at $1.00	$1.57
0.44 cubic yards sand " 0.75	0.33
0.88 " " stone " 1.50	1.32
Total materials	$3.22

The sand and stone for the 1: 4: 8 Portland mixture will cost, on the other hand, per cubic yard of concrete:

0.48 cubic yards sand at $0.75	$0.36
0.96 " " stone " 1.50	1.44
Cost of sand and stone	$1.80

Subtracting $1.80 from $3.22 leaves a difference of $1.42 which may be paid for the Portland cement in one cubic yard of concrete, and since by the quantity table 0.85 barrels of cement are required for a cubic yard of 1: 4: 8 concrete, the price for the Portland cement may be $1.42 ÷ 0.85 = $1.67 per barrel.

If the Natural cement had cost $1.25 per barrel, the price which could have been paid for Portland would have been approximately 25% higher or $2.09 per barrel.

This determination may be expressed in a formula:

$$x=\frac{am+bn+cr-(b'n+c'r)}{a'}$$

in which a, b, and c represent respectively the quantities of cement, sand, and stone required for a cubic yard of the Natural cement concrete, and m, n, and r their respective unit costs, while a', b', and c' represent similar

quantities for the Portland cement concrete, and x the required price per barrel of the Portland cement.

The following table is made out on this basis.

Prices of Portland Cement to produce Mortar or Concrete of equal cost to that from Natural Cement at $1.00 per barrel. (See p. 44.)

Proportions of Natural Cement Mortar	PROPORTIONS OF PORTLAND CEMENT MORTAR.							Proportions of Natural Cement Concrete.	PROPORTIONS OF PORTLAND CEMENT CONCRETE.				
	1:1	1:1½	1:2	1:2½	1:3	1:3½	1:4		1:2:4	1:2½:5	1:3:6	1:4:8	1:5:10
	$	$	$	$	$	$	$		$	$	$	$	$
1:1	1.00	1.23	1.46	1.69	1.92	2.15	2.38	1:2:4	1.00	1.15	1.32	1.67	2.01
1:1½		1.00	1.18	1.37	1.55	1.74	1.92	1:2½:5		1.00	1.14	1.44	1.72
1:2			1.00	1.15	1.30	1.46	1.61	1:3:6			1.00	1.26	1.51
1:2½				1.00	1.13	1.26	1.39						
1:3					1.00	1.12	1.23						

NOTE.—When the Natural cement is higher or lower than $1.00 per barrel, multiply its cost by the figures in the table to obtain approximate corresponding cost of Portland cement with which it is compared. Values make no allowance for difference in strength or labor of laying mortar.

The equivalent prices for Portland cement in mortars will be still nearer the price for Natural if allowance is made for the difference in the labor of laying brick, which in some cases may correspond to a difference of 30 cents per barrel of cement. It is evident from the table that for mortar Portland can rarely be substituted for Natural cement without increasing the cost of the work. A field still open for investigation is the employment as a substitute for Natural cement of Portland cement mixed with slaked lime or hydrated lime. The lime is so finely divided that it fills the voids and thus permits the use of more sand.

SELECTION OF THE BRAND

A precise comparison of costs of different brands of the same class of cement is impossible without thorough laboratory tests, described in Chapter VII, page 63. If the choice lies between two cements both of which have been found to be sound (see p. 77) and to set up properly, the degree of fineness, which may be readily ascertained with two sieves as described on page 67, is an aid to the decision. The finer cement will usually produce the stronger mortar.

The cheapest cement is not always the most economical. A method of comparing the relative economy of cements offered by bidders at different prices is illustrated in the following table for which the authors are indebted

to Mr. D. M. Andrews. Ten brands of Portland cement were submitted to the Government at prices ranging from \$2.77 to \$3.29.* Experiments showed that sample No. 5 was the strongest, with No. 4 a close second. The relative strength of the different brands in proportions 1:3, based on the strongest as 100.0, is given in the column headed Relative Strength of Mortar, and the column following this gives the product of the relative strength multiplied by the relative cheapness. In the case under consideration brand No. 5 was selected for purchase, because, although No. 4 gave higher economy, it appeared slightly unsound. Other data with reference to each brand was observed, including the volumes of the barrels, their gross net weights, the percentages of water used in mixing the pastes and mortar, the time of setting of the mortar, and the strength and relative economy of mortars with sand proportioned to price of cement, that is, for example, using 19% more sand with cement No. 10 than with No. 1, because the former's price was 19% greater.

*The price of Portland cement has since been materially lowered.

Relative Economy of Different Priced Portland Cements.

By D. M. Andrews.

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†Accepted in preference to No. 4 because air pat slightly defective.
‡Cement not yet set
§Based on the highest, No. 5, as 100.0.

CHAPTER V

CLASSIFICATION OF CEMENTS.

From an engineering standpoint, limes and cements may be classified as

Portland cement.
Natural cement.
Puzzolan cement.
Hydraulic lime.
Common lime.

Typical analyses of each of these are presented in the following table. The composition of Natural cement, even different samples of the same brand, is so extremely variable that their analyses cannot be regarded as characteristic of locality.

Typical Analyses of Cements.

	PORTLAND CEMENT		NATURAL CEMENT					Puzzolan Cement[7]	Hydraulic Lime (Le Tiel)[8]	COMMON LIME	
			AMERICAN		ENGL'H	FRENCH					
	Lehigh Valley[1] (mixed rock)	Western[2] (marl and clay)	Eastern Rosendale[3]	Western Louisville[3]	Roman[4]	Vassy[5]	Grappiers[6]			Lime[9]	Magnesian Lime[10]
Silica Si O_2	21.31	21.93	18.38	20.42	25.48	22.60	26.5	28.95	21.70	1.03	1.12
Alumina Al_2 O_3	6.89	5.98	15.20	4.76	10.30	8.90	2.5	11.40	3.19	1.27	0.68
Iron Oxide Fe_2 O_3	2.53	2.35		3.40	7.44	5.30	1.5	0.54	0.66		
Calcium Oxide Ca O	62.89	62.92	35.84	46.64	44.54	52.69	63.0	50.29	60.70	97.02	58.51
Magnesian Oxide Mg O	2.64	1.10	14.02	12.00	2.92	1.15	1.0	2.96	0.85	0.68	39.69
Sulphuric Acid S O_3	1.34	1.54	0.93	2.57	2.61	3.25	0.5	1.37	0.60		
Loss on Ignition	1.39	2.91	3.73	6.75	3.68	6.11	5.0	3.39	12.20		
Other constituents	0.75		11.46	3.74	1.46			0.30	0.10		

[1] W. F. Hillebrand, Society of Chemical Industry, 1902, Vol. XXI.
[2] W. F. Hillebrand, Journal American Chemical Society, 1903, 25, 1180.
[3] Clifford Richardson, *Brickbuilder*, 1897, p. 229.
[4] Stanger & Blount, Mineral Industry, Vol. V, p. 69.
[5] Candlot, Ciments et Chaux Hydrauliques, 1898, p. 174.
[6] Le Chatelier, Annales des Mines, September and October, 1893, p. 36.
[7] Report of the Board of U. S. Army Engineers on Steel Portland Cement, 1900, p. 52.
[8] Candlot, Ciments et Chaux Hydrauliques, 1898, p. 24.
[9] Rockland-Rockport Lime Co.
[10] Western Lime and Cement Co.

PORTLAND CEMENT

Portland cement is defined by Mr. Edwin C. Eckel of the U. S. Geological Survey as follows: "By the term Portland cement is to be understood the material obtained by finely pulverizing clinker produced by burning to semi-fusion an intimate artificial mixture of finely ground calcareous and argillaceous materials, this mixture consisting approximately of 3 parts of lime carbonate to 1 part of silica, alumina and iron oxide."

The definition is often further limited by specifying that the finished product must contain at least 1.7 times as much lime, by weight, as of silica, alumina, and iron oxide together.

The only surely distinguishing test of Portland cement is its chemical analysis and its specific gravity. (See pp. 64 and 65.) In the field it may often be recognized by its cold bluish gray color (see p. 113), although the color of Puzzolan and of some Natural cements is so similar that this is by no means a positive indication.

The term **Natural Portland Cement** arose from the discovery in Boulogne-sur-Mer, France, as early as 1846, of a natural rock of suitable composition for Portland cement. A similar discovery in Pennsylvania gave rise to the same term in America, but the manufacturers soon found it necessary to add to the cement rock a small percentage of purer limestone. Since the chemical composition of Portland cement, as defined above, is substantially uniform regardless of the materials from which it is made, in the United States the terms "natural" and "artificial" are meaningless.

In France, cements intermediate between Roman and Portland are called "natural Portlands."*

Sand Cement. Sand or silica cement is a mechanical mixture of Portland cement with a pure, clean sand very finely ground together in a tube mill or similar machine. For the best grades the proportions of cement to sand are 1:1, although as lean a mixture as 1:6 has been made to compete with Natural cements. The coarser particles in any Portland cement have little cementitious value, hence if a portion of the cement is replaced by inert matter and the whole ground extremely fine, its advocates maintain that the product is scarcely inferior to the unadulterated article. As made in the United States, the mixture is ground so fine that 95% of it will pass through a sieve having 200 meshes to the linear inch, and all of the 5% of residuum is said to be sand. In other words, all of the cement passes a No. 200 sieve.

*Candlot's Ciments et Chaux Hydrauliques, 1898, p. 164.

NATURAL CEMENT

Natural cement is "made by calcining natural rock at a heat below incipient fusion, and grinding the product to powder."* Natural cement contains a larger proportion of clay than hydraulic lime, and is consequently more strongly hydraulic. Its composition is extremely variable on account of the difference in the rock used in manufacture.

Natural cements in the United States in numerous instances bear the names of the localities where first manufactured. For example, Rosendale cement, a term heard in New York and New England more frequently than Natural cement, was originally manufactured in Rosendale, Ulster County, N. Y. Louisville cement first came from Louisville, Ky. The James River, Milwaukee, Utica, and Akron are other Natural cements named for localities.

The United States produces a few brands of "Improved Natural Hydraulic Cement," intermediate in quality between Natural and Portland, by mixing inferior Portland cement with Natural cement clinker.

In England the best known Natural cement is called Roman cement. Occasionally one hears the term Parker's Cement, so called from the name of the discoverer in England.

LE CHATELIER'S CLASSIFICATION OF NATURAL CEMENTS

In France there are several classes of Natural cement. Mr. H. Le Chatelier† classifies Natural cements as those obtained "by the heating of limestone less rich in lime than the limestone for hydraulic lime. They may be divided into three classes:

"Quick-setting cements, such as Vassy and Roman (Ciments à prise rapide, Vassy, romain);
"Slow-setting cements (Ciments à prise demi-lente);
"Grappiers cement (Ciments de grappiers).

"**Vassy Cements** are obtained by the heating of limestone containing much clay, at a very low temperature, just sufficient to decarbonate the lime. They are characterized by a very rapid set, followed afterwards by an extremely slow hardening, much slower than that of Portland cements."

"They differ from Portland cements by containing a much higher percentage of sulphuric acid, which appears to be one of their essential elements, and a much lower percentage of lime.

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*Professional Papers, No. 28, U. S. Army Engineers, p. 33.

†Procédés d'Essai des Matériaux Hydrauliques, Annales des Mines, 1893.

"**Slow-Setting Cements**, by the high temperature of calcination, approach Portland cements, but the natural limestones never possess the homogeneity of artificial mixtures, so that it is impossible to avoid in these cements the presence of a large quantity of free lime." The composition of these products varies from that of the Vassy cements to that of the real Portlands.

"**Grappiers Cements** are obtained by the grinding of particles which have escaped disintegration in the manufacture of hydraulic limes. These grappiers are a mixture of four distinct materials, two of which, completely inert, are unburned limestone and the clinkers formed by contact with the siliceous walls of furnaces, and two of which, strongly hydraulic, are unslaked lime and true slow-setting cement. It is necessary that the latter should predominate in the grappiers for their grinding to give a useful product. The grappier of cement is obtained regularly only by the heating of a limestone but slightly aluminous and containing about three equivalents of carbonate of lime for one of silica; its production necessitates a heating at high temperature.

"These grappiers cements are even more apt to contain free lime than the Natural cements of slow set which are obtained by the heating of limestone containing much more alumina. Because of their constitution, also, the grappiers cements may vary greatly in composition since they are produced by the grinding of a mixture of grains of cement and of various inert materials. The cement grains have very nearly the composition of tricalcium silicate (SiO_2 3 CaO)."

PUZZOLAN OR SLAG CEMENT

Puzzolan cement is the product resulting from mixing and grinding together in definite proportions slaked lime and granulated blast furnace slag or natural puzzolanic matter (such as puzzolan, santorin earth, or trass obtained from volcanic tufa).

The ancient Roman cements belonged to the class of Puzzolans. They were made by mechanically mixing slaked lime with natural puzzolana formed from the fusion of natural rock found in the volcanic regions of Italy. In Germany, trass, a volcanic product related to Puzzolan, has been used with lime in the manufacture of cements.

Blast furnace slag is essentially an artificial puzzolana, formed by the combustion in a blast furnace, and the puzzolan or slag cements of the United States are ground mixtures of granulated blast furnace slag, of special composition, and slaked lime.

A Board of Engineers officers, U. S. A., presented in 1901 the following conclusions,* based, undoubtedly, on the exhaustive studies upon the subject made by a previous Board† having the same chairman, Major W. L. Marshall:

> This term (slag or Puzzolan cement) is applied to cement made by intimately mixing by grinding together granulated blast-furnace slag of a certain quality and slaked lime, without calcination subsequent to the mixing. This is the only cement of the Puzzolan class to be found in our markets (often branded as Portland), and as true Portland cement is now made having slag for its hydraulic base, the term "slag cement" should be dropped and the generic term Puzzolan be used in advertisements and specifications for such cements.
>
> Puzzolan cement made from slag is characterized physically by its light lilac color; the absence of grit attending fine grinding and the extreme subdivision of its slaked lime element; its low specific gravity (2.6 to 2.8) compared with Portland (3 to 3.5); and by the intense bluish green color in the fresh fracture after long submersion in water, due to the presence of sulphides, which color fades after exposure to dry air.
>
> The oxidation of sulphides in dry air is destructive of Puzzolan cement mortars and concretes so exposed. Puzzolan is usually very finely ground, and when not treated with soda sets more slowly than Portland. It stands storage well, but cements treated with soda to quicken setting become again very slow setting, from the carbonization of the soda (as well as the lime) element after long storage.
>
> Puzzolan cement properly made contains no free or anhydrous lime, does not warp or swell, but is liable to fail from cracking and shrinkage (at the surface only) in dry air.
>
> Mortars and concretes made from Puzzolan approximate in tensile strength similar mixtures of Portland cement, but their resistance to crushing is less, the ratio of crushing to tensile strength being about 6 to 7 to 1 for Puzzolan, and 9 to 11 to 1 for Portland. On account of its extreme fine grinding Puzzolan often gives nearly as great tensile strength in 3 to 1 mixtures as neat.
>
> Puzzolan permanently assimilates but little water compared with Portland, its lime being already hydrated. It should be used in comparatively dry mixtures well rammed, but while requiring little water for chemical reactions, it requires for permanency in the air constant or continuous moisture.

Puzzolanic material has been suggested by Dr. Michaelis, of Germany, and Mr. R. Feret, of France (see Chapter XVIII), as a valuable addition to Portland cement designed for use in sea water.

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*Professional Papers No. 28, p. 28.

†Report of the Board of U. S. Army Engineers on Steel Portland Cement, 1900, p. 52.

HYDRAULIC LIME

The hydraulic properties of a lime, — its ability to harden under water, — are due to the presence of clay, or, more correctly, to the silica contained in the clay. Hydraulic lime is still used to quite an extent in Europe, especially in France, as a substitute for hydraulic cement. The celebrated lime-of-Teil of France is a hydraulic lime.

Mr. Edwin C. Eckel states* that "theoretically the proper composition for a hydraulic limestone should be calcium carbonate 86.8%, silica 13.2%. The hydraulic limestones in actual use, however, usually carry a much higher silica percentage, reaching at times to 25%; while alumina and iron are commonly present in quantities which may be as high as 6%. The lime content of the limestones commonly used varies from 55% to 65%."

Although the chemical composition of hydraulic lime is similar to Portland cement, its specific gravity is much lower, lying between 2.5 and 2.8.†

In the manufacture of hydraulic lime the limestone of the required composition is burned, generally in continuous kilns, and then sufficient water is added to slake the free lime produced so as to form a powder without crushing.

COMMON LIME

The commercial lime of the United States is "quicklime," which is chiefly calcium oxide (CaO).

Lime is now manufactured by a continuous process. Limestone of a rather soft texture, so as to be as free as possible from silica, iron and alumina, is charged into the top of a kiln which may be, say, 40 ft. high by 10 ft. in diameter. The fuel is introduced into combustion chambers near the foot of the shaft, and the finished product is drawn out from time to time through another opening in the bottom of the shaft. The temperature of calcination may range from 1400° Fahr. (760° Cent.) to, at times, 2,000° Fahr. (1,090° Cent.). The product (see analysis, p. 47), in ordinary lime of the best quality, is nearly pure calcium oxide (CaO). Upon the addition of water the lime slakes, forming calcium hydrate (CaH_2O_2), and, with the continued addition of water, increases in bulk to twice to three times the original loose and dry volume of the lump lime as measured in the cask. In this plastic condition it is termed by plasterers "putty" or "paste."

The setting of lime mortar is the result of three distinct processes which, however, may all go on more or less simultaneously. First, it

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**American Geologist,* March, 1902, p. 152.

†Candlot's Ciments et Chaux Hydrauliques, 1898, p. 26.

dries out and becomes firm. Second, during this operation, the calcic hydrate, which is in solution in the water of which the mortar is made, crystallizes and binds the mass together. Hydrate of lime is soluble in 831 parts of water at 78° Fahr; in 759 parts at 32° and in 1136 parts at 140°. Third, as the per cent. of water in the mortar is reduced and reaches five per cent., carbonic acid begins to be absorbed from the atmosphere. If the mortar contains more than five per cent. this absorption does not go on. While the mortar contains as much as 0.7 per cent. the absorption continues. The resulting carbonate probably unites with the hydrate of lime to form a sub-carbonate, which causes the mortar to attain a harder set, and this may finally be converted to carbonate. The mere drying out of mortar, our tests have shown, is sufficient to enable it to resist the pressure of masonry, while the further hardening furnishes the necessary bond.*

Magnesian Limes evolve less heat when slaking, expand less, and set more rapidly than pure lime. A typical analysis is given on page 47.

Hydrated Lime is a powdered slaked lime (calcium hydrate). It is manufactured by treating finely ground common lump lime with water of a certain temperature, and then cooling and screening it through a very fine screen.

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*The authors are indebted to Mr. Clifford Richardson for this paragraph.

CHAPTER VI

CHEMISTRY OF HYDRAULIC CEMENTS*

BY SPENCER B. NEWBERRY

INTRODUCTION

Hydraulic cements are compounds consisting chiefly of lime, silica, and alumina, which have the property, when mixed to a paste with water, of hardening to a stone-like mass. They may be classified as follows:

1. **Portland cement,** made by calcining at high heat an artificial mixture of carbonate of lime and clay or slag, in exactly correct proportions, and grinding the resulting clinker to powder.
2. **Natural cement,** made by burning at low heat limestone containing excess of clay and usually much magnesia, and grinding the product to powder.
3. **Hydraulic lime,** obtained by burning limestone containing a small amount of clay, slaking by sprinkling with water, and bolting the product.
4. **Puzzolan or slag cement,** consisting of a mixture of certain kinds of volcanic scoria, or of blast furnace slag, and slaked lime, ground together.

Each of these classes of cement shows peculiar qualities, and each may have advantages for certain purposes. Puzzolan cement is that used by the Romans, and many striking examples of its durability are seen in ancient structures. Slag cement, a mechanical mixture of slag and slaked lime, is made to a considerable extent in this country, and finds extended use for mortar and in work in which the greatest strength and hardness are not required. Hydraulic lime is made chiefly in France, and is but little known in the United States. Natural cement is manufactured on a very large scale from limestones containing a large proportion of clay. It is usually quick-setting, and the better qualities gain very good strength at long periods. Owing to its cheapness it is extensively used, chiefly as mortar for brickwork and masonry. All these earlier hydraulic materials, however, have gradually given way before the advance of Portland cement, as this product has been improved in quality and manufactured on a constantly increasing scale.

Portland cement was first made in England in 1827, and named from the

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*The authors are indebted to Mr. Newberry for this chapter, which has been especially prepared by him for this Treatise.

resemblance in color of the hardened cement to the building stone quarried at the Island of Portland.

MATERIALS*

As above stated, hydraulic lime and natural cements are made by burning natural limestones containing suitable amounts of clay. Portland cement, on the other hand, is made from an artificial mixture of materials, of exactly correct composition. Limestones containing clay are of frequent occurrence. If a deposit of stone containing exactly the right amount of clay, and of exactly uniform composition, could be found, Portland cement could be made from it simply by burning and grinding. For good results, however, the composition of the raw material must be *exact*, and the proportion of carbonate of lime in it must not vary even by one per cent. No natural deposit of rock of exactly this correct and unvarying composition is known or likely ever to be found; therefore Portland cement is always made from an artificial mixture, usually, if free from organic matter, containing about 75% carbonate of lime and 25% clay.

For the manufacture of Portland cement the materials chiefly used are limestone, chalk or marl, and clay. In southeastern Pennsylvania and western New Jersey occurs an unlimited deposit of *cement rock*, which consists of a slate-like limestone containing usually rather more clay than is required for a correct mixture. This is largely used for Portland cement manufacture, and is generally ground with a small amount of purer limestone to bring it to correct composition. At some of the factories in that section a correct mixture is obtained by grinding together, in suitable proportions, the upper and lower layers of the quarry. In the Central States, pure limestone, or marl (a soft and finely divided form of carbonate of lime) and clay, are the materials employed. Whatever the materials used, the first stage of the process is the preparation of an intimate and finely ground mixture of carbonate of lime and clay, of a certain definite composition, and if this is accomplished the resulting cement will be the same, whatever the original materials may have been. Success in Portland cement manufacture depends, more than upon all other features of the process, in extremely fine grinding of the raw materials. Most of the faults found in inferior Portland cement are due to neglect in this regard. Either the wet or dry process may be used in preparing the mixture. The material is then dried and calcined at white heat, generally in revolving cylindrical kilns, from which it issues in the form of small, black, rounded fragments of clinker. By grinding this clinker to fine powder the finished Portland cement is obtained.

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*The materials for cement and the manufacture of cement are also treated in Chapter XXVIII.

Magnesia in Portland cement, beyond a small percentage, has generally been considered objectionable. But little positive evidence on this point is, however, available. A committee of the German Portland Cement Manufacturers Association, many years ago, reported that magnesia up to 8 per cent. is harmless. Dyckerhoff, a member of the committee, presented a minority report stating that he had found more than 4 per cent. injurious. The subject was referred to another committee, in 1896, but this committee laid out a program of work which proved impracticable to complete, and nothing further has been accomplished. Van Blaese, in the *Thonindustriezeitung*, 1899, page 213, published a long series of tests of cements containing variable proportions of magnesia, which show that cement containing 8 per cent. is faultless, while that containing 15 per cent. is defective. The writer has made a similar series of experiments and has found that properly prepared cement with 9 per cent. magnesia passes the boiling test perfectly, while that with 15 per cent. magnesia shows expansion cracks after several hours boiling. Comparative tests of tensile strength and expansion of bars of these cements, over long periods, are now in progress. From the evidence now available it appears that the presence of magnesia up to 8 per cent., in a properly prepared Portland cement, is no disadvantage.

Sulphate of lime, in quantities exceeding about 2 per cent., is objectionable in the raw material, owing to liability of reduction to sulphide, causing the cement to turn dark blue in hardening and to give poor tests, especially with sand. This fault is more frequent with cement burned in vertical kilns than in those of the rotary type, since the former are more liable to imperfect draft and consequent reducing action.

Clay for Portland cement manufacture should be highly siliceous and practically free from coarse sand. Siliceous clays, in which the silica is from 2.5 to 3.0 times the sum of alumina and iron oxide, give mixtures which stand the high heat of the kiln without fusing, produce a clinker which is comparatively easy to grind, and yield slow-setting cements which show steady gain in strength over long periods. More aluminous clays give hard, fusible clinker and quick-setting cement, and are in many respects troublesome to use. Highly aluminous cements are believed to be especially severely attacked by sea water.

Alkalies (potash and soda) appear to exert very little influence, in the small amounts present in ordinary clays, on the character of burning or quality of the resulting cement. Excess of alkalies is believed to make cement unsound.

PROPORTION OF INGREDIENTS

Although Portland cement has been manufactured since 1827, definite rules for proportioning the ingredients have only lately been established, and are yet by no means generally accepted. In Germany it has been customary to adjust the ingredients, as recommended by Michaelis, so that the "hydraulic modulus," the ratio by weight of lime to silica, alumina and iron oxide, shall be from 1.8 to 2.2. It has, however, become generally recognized by cement chemists that much more lime combines with silica than with alumina or iron oxide. The "hydraulic modulus" is therefore a variable, and must be much higher in the case of siliceous materials than with those high in alumina and iron.

A clear explanation of the composition of Portland cement clinker was first given by Le Chatelier in 1887. From microscopic examination of clinker and hardened cement he came to the conclusion that the chief constituent of Portland cement is tri-calcium silicate, $3CaO.SiO_2$, which is the active element in the hardening. This tri-silicate is produced by chemical precipitation from a mass of a multiple silico-aluminate which serves as a vehicle for the silica and lime and permits their combination, but remains inert during the hardening. Le Chatelier stated that the lime and magnesia in Portland cement should not exceed a maximum,

$$\frac{CaO+MgO}{SiO_2+Al_2O_3} \leq 3 \qquad (1)$$

nor be less than a minimum,

$$\frac{CaO+MgO}{SiO_2-Al_2O_3-Fe_2O_3} \geq 3 \qquad (2)$$

These formulas represent chemical equivalents and not weights.

The best brands of modern Portland cement approach pretty closely to the above maximum formula, while one corresponding to the minimum formula would be so greatly over-clayed as to be practically useless.

The hardening of cement, according to Le Chatelier, consists in the decomposition of the tri-silicate by water, with the formation of crystalline calcium hydrate and hydrated mono-silicate.

Since the publication of the above researches the constitution of clinker and hardened cement have been investigated by numerous experimenters, and a great number of new theories have been propounded. It cannot be said, however, that any of Le Chatelier's important statements have been disproved, nor that any material advance has been made upon the theory which he proposed. At the present time Portland cement clinker is re-

garded by nearly all cement chemists as a crystalline mass of tri-calcium silicate, imbedded in a non-crystalline magma consisting of a fusible compound of silica and lime with practically all the alumina and iron oxide.

Le Chatelier's formulas are inconvenient in form and incomprehensible except to those familiar with chemical formulas. The writer published in 1897 (*Journal of the Society of Chemical Industry*, Nov. 30, 1897) a paper on the constitution of hydraulic cements, containing an account of a series of experiments based on the work of Le Chatelier. It was found that the maximum of lime which could be brought into combination to produce a sound cement is three molecules for each molecule of silica present, and two molecules for each molecule of alumina. The composition of cement containing the maximum of lime would therefore be expressed by the formula

$$X(3CaO.SiO_2) + Y(2CaO.Al_2O_3) \qquad (3)$$

It is understood that this formula is merely empirical, representing the relative proportions present, since the aluminate remains for the most part in the magma in combination with part of the silica and with other substances.

Substituting weights for equivalents, the above formula may be expressed as follows:

$$\text{Lime} = \text{silica} \times 2.8 + \text{alumina} \times 1.1.$$

It should be remembered that this formula represents the ***maximum*** of lime which a Portland cement, burned in the usual manner, may contain without showing unsoundness. This maximum can be reached only by extremely fine grinding of the raw material. This formula, also, by no means represents the composition of finished cement, since the ash of the fuel lowers the lime and raises the silica and alumina, above that calculated from the raw material, by at least 2 per cent.

In the laboratory, using gas as fuel, it will be found practicable to prepare sound cements corresponding to the above formula. In actual manufacture it is safer to reduce the lime slightly, to counterbalance possible defective grinding of raw material or unavoidable variations in composition. It will be found that the raw material at factories where the best Portland cements are made rarely falls below the composition,

$$\text{Lime} = \text{silica} \times 2.7 + \text{alumina} \times 1.0. \qquad (4)$$

This may be taken as a safe practical formula for commercial use. With fine grinding of the raw material it will invariably yield sound cements,

while the use of a lower proportion of lime will be likely to produce quick-setting cement, low in tensile strength. As already explained, commercial cements are considerably lower in lime, owing to change in composition produced by the fuel-ash.

The writer's experiments have shown that magnesia forms with clay no products having hydraulic properties. It should therefore be disregarded in calculating cement mixtures, the composition of which should be calculated on the basis of the silica, alumina and lime only, without regard to the magnesia present. Iron oxide, also, in the quantities usually met with in ordinary clays, plays an insignificant part so far as the proportions of the constituents are concerned, and may be disregarded in the calculation.

As a practical example of the use of the above formula, let us suppose that we wish to make cement from limestone and clay of the following composition:

	Limestone	Clay
Lime	52.6	2.2
Magnesia	0.7	1.9
Silica	3.2	65.4
Alumina	1.0	16.5
Iron Oxide	0.3	6.1
Loss on ignition, etc.	42.2	7.9
	100.0	100.0

The silica and alumina in the limestone will require

3.2 × 2.7 + 1.0 = 9.6% lime, leaving 52.6 — 9.6 = 43.0% lime available for combination with clay.

The silica and alumina in 100 parts clay will require

65.4 × 2.7 + 16.5 × 1.0 = 193.1 parts lime. Subtracting the lime contained in the clay we have

193.1 — 2.2 = 190.9 parts lime required for 100 parts clay.

As the 100 parts stone contain 43 parts available lime, that amount of stone will require

$$\frac{43 \times 100}{190.9} = 22.5 \text{ parts clay.}$$

The composition of the charge and of the resulting cement may be tabulated as follows:

	100 STONE	22.5 CLAY	122.5 MIX	78.52 CEMENT	100 CEMENT
Lime	52.60	0.50	53.10	53.10	67.63
Magnesia	0.70	.43	1.13	1.13	1.44
Silica	3.20	14.71	17.91	17.91	22.81
Alumina	1.00	3.71	4.71	4.71	6.00
Iron Oxide	0.30	1.37	1.67	1.67	2.12
Loss, etc.	42.20	1.78	43.98	....	....
	100.00	22.50	122.50	78.52	100.00

As stated above, the ash of the fuel will change the composition of the resulting cement materially; analysis of the product, burned with coal, will probably show about 65 per cent. lime and perhaps 24 per cent. silica. This fuel-ash is, however, not uniformly distributed through the product, but attaches itself chiefly to the surfaces of the clinker. It is not, therefore, found practicable to materially raise the proportion of lime to counterbalance the silica and alumina of the ash.

It will be noted that in the above calculated analysis of raw mixture and cement the

$$\frac{\text{Lime—alumina}}{\text{silica}} = 2.7$$

The writer proposes to call this figure the *lime factor* of the mixture. Adoption of this factor will give cements of practically maximum quality with any materials, whether siliceous or aluminous, provided the mix is finely ground and properly burned. Owing to the influence of the ash of the fuel, as above explained, the factor of finished cements will be found about 0.2 lower than that of the raw material. The best commercial cements generally show a factor of 2.5 to 2.6, though made from mixtures with a factor of 2.7 to 2.8.

The following analyses, taken from a paper by the writer in *Cement and Engineering News*, November, 1901, show the influence of the fuel-ash on the composition of the clinker. The samples of clinker were taken one

hour later than those of raw material, since the passage through the kiln required about one hour.

Lehigh Portland Cement Co., Allentown, Pa.

	Mix	Clinker, calculated from mix	Clinker found
SiO_2	14.33	22.18	22.96
Al_2O_3	4.32	6.68	6.78
Fe_2O_3	1.46	2.26	2.54
CaO	42.69	66.08	63.95
MgO and SO_3	1.81	2.80	2.94
Loss	35.14	....	....
	99.75	100.00	99.17
Factor $\frac{CaO - Al_2O_3}{SiO_2}$	....	2.68	2.49

Sandusky Portland Cement Co., Syracuse, Ind.

	Mix	Clinker, calculated from mix	Clinker found
SiO_2	13.50	22.02	22.33
Al_2O_3	3.43	5.60	5.53
Fe_2O_3	1.27	2.07	3.28
CaO	40.76	66.49	64.40
MgO and SO_3	3.27	3.82	3.61
Loss	38.30	....	....
	100.53	100.00	99.15
Factor $\frac{CaO - Al_2O_3}{SiO_2}$	....	2.76	2.63

Comparison of the above analyses of mix and clinker shows how greatly the ash of the fuel affects the composition. In commercial cement a still further reduction in the proportion of lime is caused by the addition of gypsum and the absorption of moisture and carbonic acid from the air. It will be readily seen, therefore, that analysis of finished cement gives but little indication of the true proportion of ingredients or of the quality of the product.

EFFECT OF COMPOSITION ON QUALITY

Too high proportion of lime (lime factor of mix above 2.8) will give a slow-setting cement which will fail in the boiling test. If the excess of lime is great, pats of cement kept in cold water will show radial expansion cracks at the edges after a certain time, perhaps even within a few days. The same defects result from *imperfect grinding of the raw material*, and are far more often due to this cause than to excess of lime. Cement which is unsound and shows expansion from either cause may be improved and perhaps made sound by storage or by exposure to air. It is not, however, safe to rely greatly on this remedy. Lack of soundness is in all cases due to faulty manufacture, since well-burned cement made from suitably prepared raw material will invariably pass all soundness tests when fresh from the grinding mills. Consumers are advised to accept no cement which fails to pass a reasonable boiling test, as they will thus err, if at all, on the safe side, and will influence careless manufacturers to improve their process.

Too low proportion of lime, giving an over-clayed mixture, produces a fusible clinker, liable to overburning. This is especially the case with aluminous materials. If hard-burned, such mixtures give a fused clinker liable to fall to dust on cooling, hard to grind, and yielding slow-setting cement of poor hardening properties. If light-burned, an over-clayed mixture yields soft brownish clinker, grinding to a brownish, quick-setting cement of inferior strength.

Overburning rarely occurs except with over-clayed mixtures or in consequence of the fluxing action of the fuel-ash or the brick lining of the kiln. Properly proportioned mixtures stand a very high heat without injury.

Underburning, as stated above, in the case of an over-clayed mixture, yields quick-setting and weak cement. Normal mixtures, when underburned, usually give cement which fails in soundness tests. Light burning is generally indicated by heating of the cement on mixing with water. This behavior generally accompanies quick setting, and may be so marked as to be quite apparent to the touch of the fingers. Some cements, though slow-setting when first made, become very quick-setting on storage. Cases are on record in which this change has taken place within a few days. After longer periods the original slow-setting quality may return. The cause of this phenomenon has not been determined; it may be said, however, that troubles of this class, including quick setting and heating with water, are especially characteristic of cements made from aluminous materials.

CHAPTER VII

STANDARD CEMENT TESTS

The tests which are regarded as most suitable for the selection and acceptance of cement for important concrete construction are as follows:

Chemical analysis.
Specific gravity.
Fineness.
Activity, or time of setting.
Tensile strength of neat cement and sand mortars.
Soundness or constancy of volume.

The French Commission* in 1893, in addition to these tests, gave standard rules for testing weight, homogeneity (with the microscope), compressive strength, bending strength, yield of paste and mortar (*rendement*), porosity, permeability, decomposition, and adhesion, one or more of which tests may be desirable under certain conditions. As these are usually of minor importance, however, special mention of them is reserved for the following chapter.

In unimportant construction it is often safe to use a first-class American Portland cement without testing, and in other cases the test for soundness is the only one which need be actually made. Under almost all circumstances, however, when purchasing cement, full specifications (see Chapter III, p. 28) are advisable, so that if the cement does not work satisfactorily it may be more carefully examined and unused portions rejected.

In this chapter are presented, in addition to the description of the methods of making cement tests, complete lists of apparatus for a large and a small laboratory (p. 80), formulas and tables for determining the quantity of water in cement mortars (p. 85), comparisons of American and European practice in cement testing, a discussion of the causes of unsoundness and the results of soundness tests (p. 101), curves showing the growth in strength of typical cements and cement mortars (p. 99), and other information with reference to the qualities and testing of Portland cement.

STANDARD METHODS OF CEMENT TESTING

The following recommendations for testing are reprinted, with comments by the authors, from the preliminary or Progress Report of Special Com-

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*Commission des Méthodes d'Essai des Matériaux de Construction, 1894, Vol. 1, p. 235.

mittee on Uniform Tests of Cement of the American Society of Civil Engineers,* as presented in 1903 and amended in 1904. The methods are designed particularly for the testing of Portland cement, but are applicable to Natural (and also to Puzzolan), with the exception of paragraphs 5, 6, 70 (2), 73 and 76.

The standards which should be attained by first-class Portland and Natural cements are presented in the Standard Specifications in Chapter III, page 28.

Sampling. 1. *Selection of Sample.* — The selection of the sample for testing is a detail that must be left to the discretion of the engineer; the number and the quantity to be taken from each package will depend largely on the importance of the work, the number of tests to be made and the facilities for making them.

2. The sample shall be a fair average of the contents of the package; it is recommended that, where conditions permit, one barrel in every ten be sampled.

3. All samples should be passed through a sieve having twenty meshes per linear inch, in order to break up lumps and remove foreign material; this is also a very effective method for mixing them together in order to obtain an average. For determining the characteristics of a shipment of cement, the individual samples may be mixed and the average tested; where time will permit, however, it is recommended that they be tested separately.

4. *Method of Sampling.* — Cement in barrels should be sampled through a hole made in the center of one of the staves, midway between the heads, or in the head, by means of an auger or a sampling iron similar to that used by sugar inspectors. If in bags, it should be taken from surface to center.

FIG. 8. Sampling Iron. (*See p.* 64.)

A sampling iron is illustrated in Fig. 8.

With the usual packing of Portland cement, four bags to the barrel, one bag in forty is equivalent to one barrel in ten. There is no necessity because of the smaller size of the packages for testing a larger proportion of the total weight.

The practice of mixing samples taken from a number of packages is by many authorities not considered advisable except for the purpose, suggested above, "of determining the characteristics of a shipment." A mixture of samples will not reveal irregularities in quality.

Chemical Analysis. 5. *Significance.* — Chemical analysis may render valuable service in the detection of adulteration of cement with considerable

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*Proceedings, January, 1903.

amounts of inert material, such as slag or ground limestone. It is of use, also, in determining whether certain constituents, believed to be harmful when in excess of a certain percentage, as magnesia and sulphuric anhydride, are present in inadmissible proportions.

6. The determination of the principal constituents of cement — silica, alumina, iron oxide and lime — is not conclusive as an indication of quality. Faulty character of cement results more frequently from imperfect preparation of the raw material or defective burning than from incorrect proportions of the constituents. Cement made from very finely ground material, and thoroughly burned, may contain much more lime than the amount usually present and still be perfectly sound. On the other hand cements low in lime may, on account of careless preparation of the raw material, be of dangerous character. Further, the ash of the fuel used in burning may so greatly modify the composition of the product as largely to destroy the significance of the results of analysis.

7. *Method.* — As a method to be followed for the analysis of cement, that proposed by the Committee on Uniformity in the Analysis of Materials for the Portland Cement Industry, of the New York Section of the Society for Chemical Industry, and published in the Journal of the Society for January 15, 1902, is recommended.*

An exceedingly simple test for determining adulteration with raw or partially burned rock, is the treatment of the cement with muriatic acid as described in the purity test on page 4. It does not furnish the percentage of foreign ingredients, but the black precipitation of the adulterant darkens the color of the yellow jelly to a degree depending upon the quantity of adulteration.

Specific Gravity. 8. *Significance.* — The specific gravity of cement is lowered by underburning, adulteration and hydration, but the adulteration must be in considerable quantity to affect the results appreciably.

9. Inasmuch as the differences in specific gravity are usually very small, great care must be exercised in making the determination.

10. When properly made, this test affords a quick check for underburning or adulteration.

11. *Apparatus and Method.* — The determination of specific gravity is most conveniently made with Le Chatelier's apparatus. This consists of a flask (*D*), Fig. 9, of 120 cu. cm. (7.32 cu. in.) capacity, the neck of which is about 20 cm. (7.87 in.) long; in the middle of this neck is a bulb (*C*), above and below which are two marks (*F*) and (*E*); the volume between these marks is 20 cu. cm. (1.22 cu. in.). The neck has a diameter of about 9 mm. (0.35 in.), and is graduated into tenths of cubic centimeters above the mark (*F*).

12. Benzine (62° Baumé naphtha), or kerosene free from water, should be used in making the determination.

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*Printed in Appendix I of this Treatise.

13. The specific gravity can be determined in two ways:

(1) The flask is filled with either of these liquids to the lower mark (*E*), and 64 grams (2.25 oz.) of powder, previously dried at 100° Cent. (212° Fahr.) and cooled to the temperature of the liquid, is gradually introduced through the funnel (*B*) [the stem of which extends into the flask to the top of the bulb (*C*)], until the upper mark (*F*) is reached. The difference in weight between the cement remaining and the original quantity (64 g.) is the weight which has displaced 20 cu. cm.

B

FIG. 9.—Le Chatelier's Specific Gravity Apparatus. (*See p.* 65.)

14. (2) The whole quantity of the powder is introduced, and the level of the liquid rises to some division of the graduated neck. This reading plus 20 cu. cm. is the volume displaced by 64 grams of the powder.

15. The specific gravity is then obtained from the formula:

$$\text{Specific Gravity} = \frac{\text{Weight of Cement}}{\text{Displaced Volume}}$$

16. The flask, during the operation, is kept immersed in water in a jar (A), in order to avoid variations in the temperature of the liquid. The results should agree within 0.01.

17. A convenient method for cleaning the apparatus is as follows: The flask is inverted over a large vessel, preferably a glass jar, and shaken vertically until the liquid starts to flow freely; it is then held still in a vertical position until empty; the remaining traces of cement can be removed in a similar manner by pouring into the flask a small quantity of clean liquid and repeating the operation.

18. More accurate determinations may be made with the picnometer.

The usual specific gravities of different classes of cement are given on page 81.

Le Chatelier's apparatus, suggested above as most convenient, was also recommended by Mr. E. Candlot after experiments for the French Commission,* in which he employed for comparison the Mann, Keate, Schumann, and Candlot, as well as the Le Chatelier apparatus.

Mr. Daniel D. Jackson† has more recently devised an apparatus with which temperature corrections can be made more readily than with the older types.

Fineness. 19. *Significance.* — It is generally accepted that the coarser particles in cement are practically inert, and it is only the extremely fine powder that possesses adhesive or cementing qualities. The more finely cement is pulverized, all other conditions being the same, the more sand it will carry and produce a mortar of a given strength.

20. The degree of final pulverization which the cement receives at the place of manufacture is ascertained by measuring the residue retained on certain sieves. Those known as the No. 100 and No. 200 sieves are recommended for this purpose.

21. *Apparatus.* — The sieves should be circular, about 20 cm. (7.87 in.) in diameter, 6 cm. (2.36 in.) high, and provided with a pan, 5 cm. (1.97 in.) deep, and a cover.

22. The wire cloth should be woven (not twilled) from brass wire having the following diameters:

No. 100, 0.0045 in.; No. 200, 0.0024 in.

23. This cloth should be mounted on the frames without distortion; the mesh should be regular in spacing and be within the following limits:

No. 100, 96 to 100 meshes to the linear inch.
No. 200, 188 to 200 " " " "

24. Fifty grams (1.76 oz.) or 100 g. (3.52 oz.) should be used for the test, and dried at a temperature of 100° Cent. (212° Fahr.) prior to sieving.

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*Commission des Méthodes d'Essai des Matériaux de Construction, 1895, Vol. IV, p. 15.

†See *Engineering Record,* July 16, 1904, p. 82.

25. *Method.* — The Committee, after careful investigation, has reached the conclusion that mechanical sieving is not as practicable or efficient as hand work, and, therefore, recommends the following method:

26. The thoroughly dried and coarsely screened sample is weighed and placed on the No. 200 sieve, which, with pan and cover attached, is held in one hand in a slightly inclined position, and moved forward and backward, at the same time striking the side gently with the palm of the other hand, at the rate of about 200 strokes per minute. The operation is continued until not more than one-tenth of 1% passes through after one minute of continuous sieving. The residue is weighed, then placed on the No. 100 sieve and the operation repeated. The work may be expedited by placing in the sieve a small quantity of large shot. The results should be reported to the nearest tenth of 1 per cent.

Laboratory scales for weighing the samples and the residue are illustrated in Fig. 10.

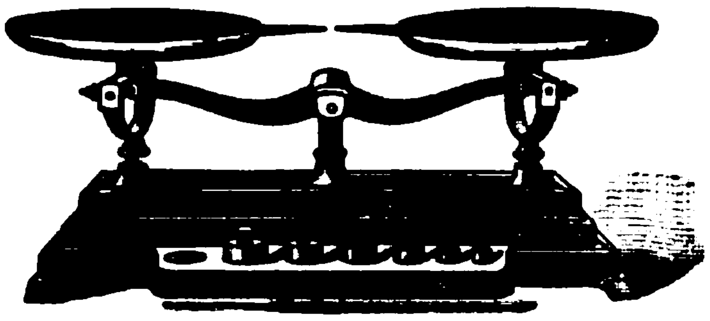

FIG. 10.—Delicate Laboratory Scales. (*See p.* 68.)

A table is given on page 84 for comparing American and European sieves, and the effect of the fineness of cement upon its strength is discussed on page 82.

It is impracticable to sift cement through a sieve finer than 200 meshes per linear inch. The particles which will just pass a No. 200 sieve are about 0.10 millimeter (0.0004 in.) in diameter.* For still further separating the cement, some method based on the principle of suspension in liquid is employed as described on page 85.

Normal Consistency. 27. *Significance.* — The use of a proper percentage of water in making the pastes† from which pats, tests of setting and briquettes are made, is exceedingly important, and affects vitally the results obtained.

28. The determination consists in measuring the amount of water required to reduce the cement to a given state of plasticity, or to what is usually designated the normal consistency.

29. Various methods have been proposed for making this determination, none of which has been found entirely satisfactory. The Committee recommends the following:

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*Allen Hazen in Report Massachusetts State Board of Health, 1892.

†The term "paste" is used in this report to designate a mixture of cement and water, and the word "mortar" a mixture of cement, sand and water.

30. ***Method. Vicat Needle Apparatus.*** — This consists of a frame (*K*), Fig. 11, bearing a movable rod (*L*), with the cap (*A*) at one end, and at the other the cylinder (*B*), 1 cm. (0.39 in.) in diameter, the cap, rod and cylinder weighing 300 grams (10.58 oz.). The rod, which can be held in any desired position by a screw (*F*), carries an indicator, which moves over a scale (graduated to centimeters) attached to the frame (*K*). The paste is held by a conical, hard-rubber ring (*I*), 7 cm. (2.76 in.) in diameter at the base, 4 cm. (1.57 in.) high, resting on a glass plate (*J*), about 10 cm. (3.94 in.) square.

31. In making the determination the same quantity of cement as will be subsequently used for each batch in making the briquettes (but not less than 500 grams) (17.64 oz.) are kneaded into a paste, as described in Paragraph 58, and quickly formed into a ball with the hands, completing the operation by tossing it six times from one hand to the other, maintained 6 in. apart; the ball is then pressed into the rubber ring, through the larger opening, smoothed off, and placed (on its large end) on a glass plate and the smaller end smoothed off with a trowel; the paste, confined in the ring, resting on the plate, is placed under the rod bearing the cylinder, which is brought in contact with the surface and quickly released.

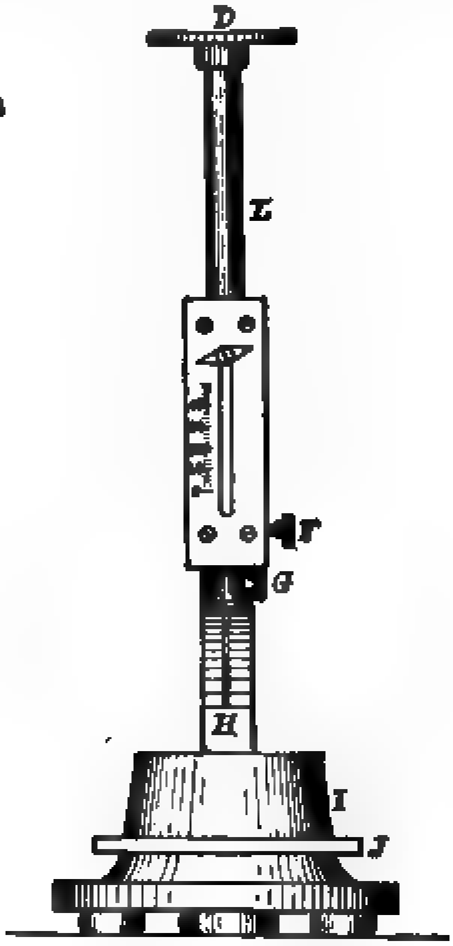

FIG. 11.—Vicat Needle. (*See p.* 69.)

32. The paste is of normal consistency when the cylinder penetrates to a

point in the mass 10 mm. (0.39 in.) below the top of the ring. Great care must be taken to fill the ring exactly to the top.

33. The trial pastes are made with varying percentages of water until the correct consistency is obtained.

34. The Committee has recommended, as normal, a paste, the consistency of which is rather wet, because it believes that variations in the amount of compression to which the briquette is subjected in molding are likely to be less with such a paste.

35. Having determined in this manner the proper percentage of water required to produce a paste of normal consistency, the proper percentage required for the mortars is obtained from an empirical formula.

36. The Committee hopes to devise such a formula. The subject proves to be a very difficult one, and, although the Committee has given it much study, it is not yet prepared to make a definite recommendation.

Formulas of Mr. R. Feret for determining the percentage of water for sand mortars, and an approximate table, are presented on pages 86 and 88.

The Boulogne Method for determining the proper consistency of neat paste was formerly in general use in France, and is still the best guide for determining the correct consistency of paste when the Vicat apparatus is not available. The Vicat needle, however, should be included in every well equipped cement laboratory, experiments by Messrs. P. Alexandre and R. Feret for the French Commission* showing that it gives much more uniform results than the Boulogne method.

The Boulogne method requires that the paste shall be firm but well bonded, shining and plastic, and shall satisfy the following conditions:

1. The consistency shall not change if it is worked 3 minutes longer than the original 5 minutes.†

2. If dropped 50 centimeters (20 in.) from a trowel, it should leave the trowel clean, and fall without losing its shape or cracking.

3. Light pressure in the hand should bring water to the surface, and the paste should not stick to the hand. If a ball thus formed falls from a height of about 50 centimeters (20 in.) it should retain its rounded form without showing cracks.

4. The proportion of water should be such that more or less will produce opposite effects from those just described for the proper consistency.

Time of Setting. 37. *Significance.* — The object of this test is to determine the time which elapses from the moment water is added until the paste ceases to be fluid and plastic (called the "initial set"), and also the time required for it to acquire a certain degree of hardness (called the

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*Commission des Méthodes d'Essai des Matériaux de Construction, 1895, Vol. IV, p. 49.

†The original working for the U. S. Standard tests is 1½ minutes (see paragraph 58).

"final" or "hard set"). The former of these is the more important, since, with the commencement of setting, the process of crystallization or hardening is said to begin. As a disturbance of this process may produce a loss of strength, it is desirable to complete the operation of mixing and molding or incorporating the mortar into the work before the cement begins to set.

38. It is usual to measure arbitrarily the beginning and end of the setting by the penetration of weighted wires of given diameters.

39. *Method.* — For this purpose the Vicat Needle, which has already been described in Paragraph 30, should be used.

40. In making the test, a paste of normal consistency is molded and placed under the rod (*L*), Fig. 11, as described in Paragraph 31; this rod, bearing the cap (*D*) at one end and the needle (*H*), 1 mm. (0.039 in.) in diameter, at the other, weighing 300 g. (10.58 oz.). The needle is then carefully brought in contact with the surface of the paste and quickly released.

41. The setting is said to have commenced when the needle ceases to pass a point 5 mm. (0.20 in.) above the upper surface of the glass plate, and is said to have terminated the moment the needle does not sink visibly into the mass.

42. The test pieces should be stored in moist air during the test; this is accomplished by placing them on a rack over water contained in a pan, and covered with a damp cloth, the cloth to be kept away from them by means of a wire screen; or they may be stored in a moist box or closet.

43. Care should be taken to keep the needle clean, as the collection of cement on the sides of the needle retards the penetration, while cement on the point reduces the area and tends to increase the penetration.

44. The determination of the time of setting is only approximate, being materially affected by the temperature of the mixing water, the temperature and humidity of the air during the test, the percentage of water used, and the amount of molding the paste receives.

For practical purposes in ordinary construction where laboratory apparatus is unavailable, the setting qualities of a cement or mortar may often be examined by making up pats from a number of the packages and trying their hardening by pressure of the thumb. Where the thumb fails to indent the surface the paste or mortar may be considered to have reached its final set.

The Gillmore needles, described on page 89 and there compared with the Vicat apparatus, were formerly the U. S. standard.

Standard Sand. 45. The Committee recognizes the grave objections to the standard quartz now generally used, especially on account of its high percentage of voids, the difficulty of compacting in the molds, and its lack of uniformity; it has spent much time in investigating the various natural sands which appeared to be available and suitable for use.

46. For the present, the Committee recommends the natural sand from

Ottawa, Ill., screened to pass a sieve having 20 meshes per linear inch and retained on a sieve having 30 meshes per linear inch; the wires to have diameters of 0.0165 and 0.0112 in., respectively, *i. e.*, half the width of the opening in each case. Sand having passed the No. 20 sieve shall be considered standard when not more than one per cent. passes a No. 30 sieve after one minute continuous sifting of a 500-gram sample.

47. The Sandusky Portland Cement Company, of Sandusky, Ohio, has agreed to undertake the preparation of this sand, and to furnish it at a price sufficient only to cover the actual cost of preparation.

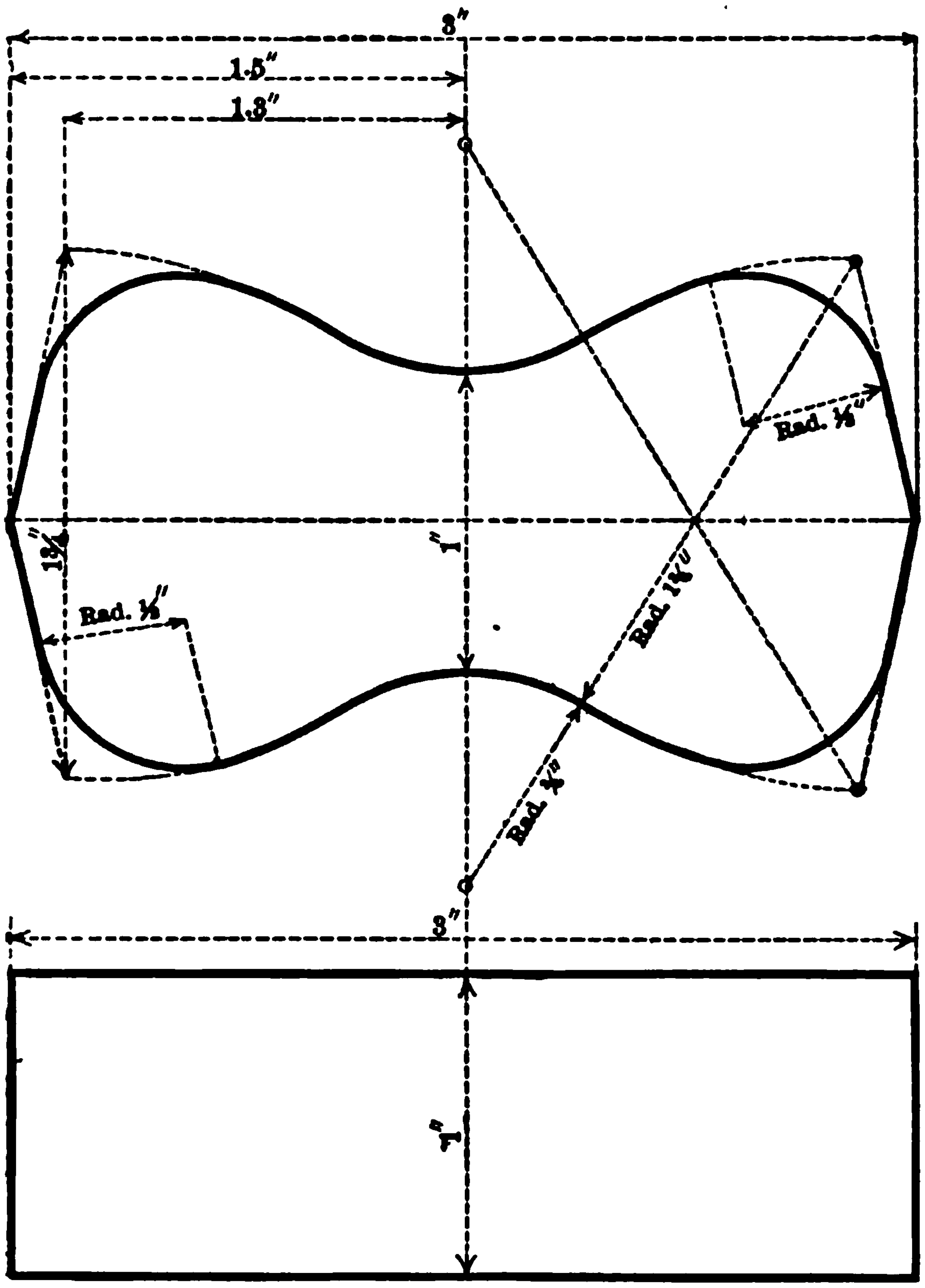

FIG. 12.—Details for Briquette. (*See p.* 72.)

Photographs of the grains of Ottawa and of crushed quartz sand are shown on page 175.

European is compared with U. S. standard sand on page 90.

Form of Briquette. 48. While the form of the briquette recommended by a former Committee of the Society is not wholly satisfactory, this Committee is not prepared to suggest any change, other than rounding off the corners by curves of ½-in. radius, Fig. 12.

The German standard briquette is sketched on page 92.

Molds. 49. The molds should be made of brass, bronze, or some equally non-corrodible material, having sufficient metal in the sides to prevent spreading during molding.

50. Gang molds, which permit molding a number of briquettes at one

time, are preferred by many to single molds; since the greater quantity of mortar that can be mixed tends to produce greater uniformity in the results. The type shown in Fig. 13 is recommended.

51. The molds should be wiped with an oily cloth before using.

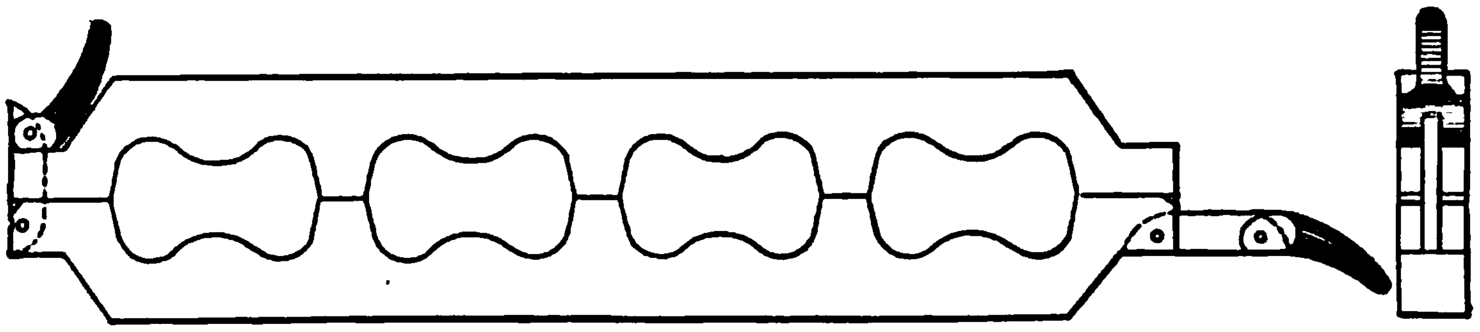

FIG. 13.—Details for Gang Mold. (*See p.* 73.)

Mixing. 52. All proportions should be stated by weight; the quantity of water to be used should be stated as a percentage of the dry material.

53. The metric system is recommended because of the convenient relation of the gram and the cubic centimeter.

54. The temperature of the room and the mixing water should be as near 21° Cent. (70° Fahr.) as it is practicable to maintain it.

55. The sand and cement should be thoroughly mixed dry. The mixing should be done on some non-absorbing surface, preferably plate glass. If the mixing must be done on an absorbing surface it should be thoroughly dampened prior to use.

56. The quantity of material to be mixed at one time depends on the number of test pieces to be made; about 1,000 gr. (35.28 oz.) makes a convenient quantity to mix, especially by hand methods.

57. The Committee, after investigation of the various mechanical mixing machines, has decided not to recommend any machine that has thus far been devised, for the following reasons:

(1) The tendency of most cement is to "ball up" in the machine, thereby preventing the working of it into a homogeneous paste; (2) there are no means of ascertaining when the mixing is complete without stopping the machine, and (3) the difficulty of keeping the machine clean.

58. *Method.* — The material is weighed and placed on the mixing table, and a crater formed in the center, into which the proper percentage of clean water is poured; the material on the outer edge is turned into the crater by the aid of a trowel. As soon as the water has been absorbed, which should not require more than one minute, the operation is completed by vigorously kneading with the hands for an additional 1½ minutes, the process being similar to that used in kneading dough. A sand-glass affords a convenient guide for the time of kneading. During the operation of mixing, the hands should be protected by gloves, preferably of rubber.

The apparatus required for mixing briquettes consists of a piece of 1-inch plate glass at least 24 inches square, counter scales (preferably metric system), recording from 10 grams to 1½ kilograms, a 250

cubic centimeter graduated measuring glass, rubber gloves, one 8-inch mason's trowel, one 4-inch pointing trowel, Fig. 14, and a thermometer.

FIG. 14.
(*See p.* 74.)

European standards specify mixing five minutes instead of one and a half minutes. This difference in time is due to the methods of manipulation, in Europe the materials being mixed with a trowel or spoon. Experiments by the authors tend to show that a denser mixture can be obtained by kneading one and a half minutes than by mixing five minutes with a trowel, so that the American method is both quicker and better.

Molding. 59. Having worked the paste or mortar to the proper consistency, it is at once placed in the molds by hand.

60. The Committee has been unable to secure satisfactory results with the present molding machines; the operation of machine molding is very slow, and the present types permit of molding but one briquette at a time, and are not practicable with the pastes or mortars herein recommended.

61. *Method.* — The molds should be filled at once, the material pressed in firmly with the fingers and smoothed off with a trowel without ramming; the material should be heaped up on the upper surface of the mold, and, in smoothing off, the trowel should be drawn over the mold in such a manner as to exert a moderate pressure on the excess material. The mold should be turned over and the operation repeated.

62. A check upon the uniformity of the mixing and molding is afforded by weighing the briquettes just prior to immersion, or upon removal from the moist closet. Briquettes which vary in weight more than 3% from the average should not be tested.

The method of introducing the paste or mortar into the molds exercises considerable effect upon the strength of the specimen. If a comparatively dry mixture is employed and it is packed in thin layers into the mold, a denser mass results and the strength is higher, especially on short-time tests, than with specimens of a wet or plastic consistency. Results from plastic cements and mortars, however, show greater uniformity.

Although the French Commission in 1893 specified the method of using dry mortar, they recommended that after an international agreement standard plastic mortars be employed for all tests.

Experiments by Mr. R. Feret, made for the French Commission,* which are summarized in an article by the authors† on *Variation in Strength of Mortars*, give the comparative strengths of specimens beaten with a spatule

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*Commission des Méthodes d'Essai des Matériaux de Construction, 1895, Vol. IV, p. 73.

†*Cement*, July, 1903, p. 165.

(the German method), pressed with a hand rammer, rammed in the Tetmajer apparatus, and rammed with the Bohme rammer (an alternate German method).

Storage of the Test Pieces. 63. During the first 24 hours after molding, the test pieces should be kept in moist air to prevent them from drying out.

64. A moist closet or chamber is so easily devised that the use of the damp cloth should be abandoned if possible. Covering the test pieces with a damp cloth is objectionable, as commonly used, because the cloth may dry out unequally, and, in consequence, the test pieces are not all maintained under the same condition. Where a moist closet is not available, a cloth may be used and kept uniformly wet by immersing the ends in water. It should be kept from direct contact with the test pieces by means of a wire screen or some similar arrangement.

65. A moist closet consists of a soapstone or slate box, or a metal-lined wooden box — the metal lining being covered with felt and this felt kept wet. The bottom of the box is so constructed as to hold water, and the sides are provided with cleats for holding glass shelves on which to place the briquettes. Care should be taken to keep the air in the closet uniformly moist.

66. After 24 hours in moist air, the test pieces for longer periods of time should be immersed in water maintained as near 21° Cent. (70° Fahr.) as practicable; they may be stored in tanks or pans, which should be of noncorrodible material.

A moist closet and storage pans designed by Mr. Richard L. Humphrey are shown in Fig. 15, page 75, and Fig. 16, page 76.

Tensile Strength. 67. The tests may be made on any standard machine. A solid metal clip, as shown in Fig. 17, is recommended. This

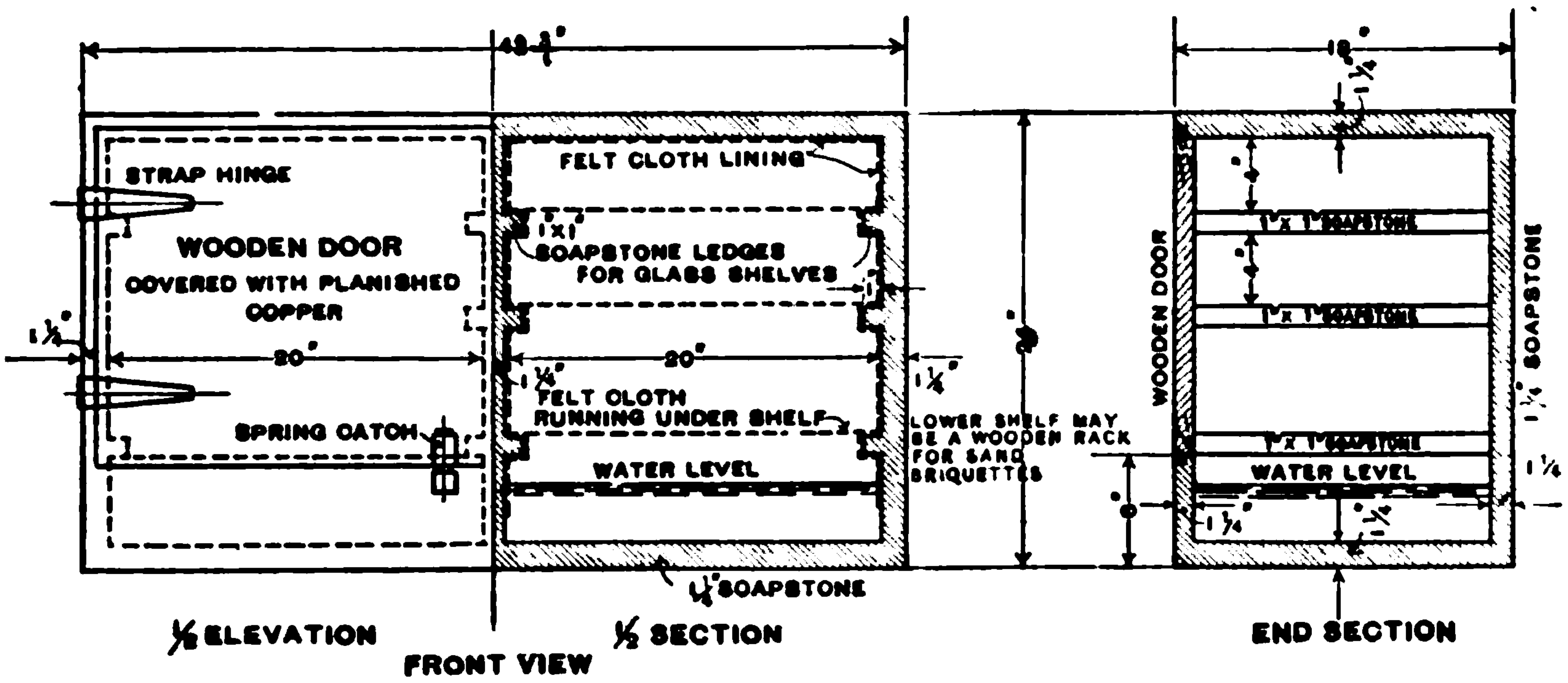

FIG. 15.—Moist Closet. (*See p.* 75.)

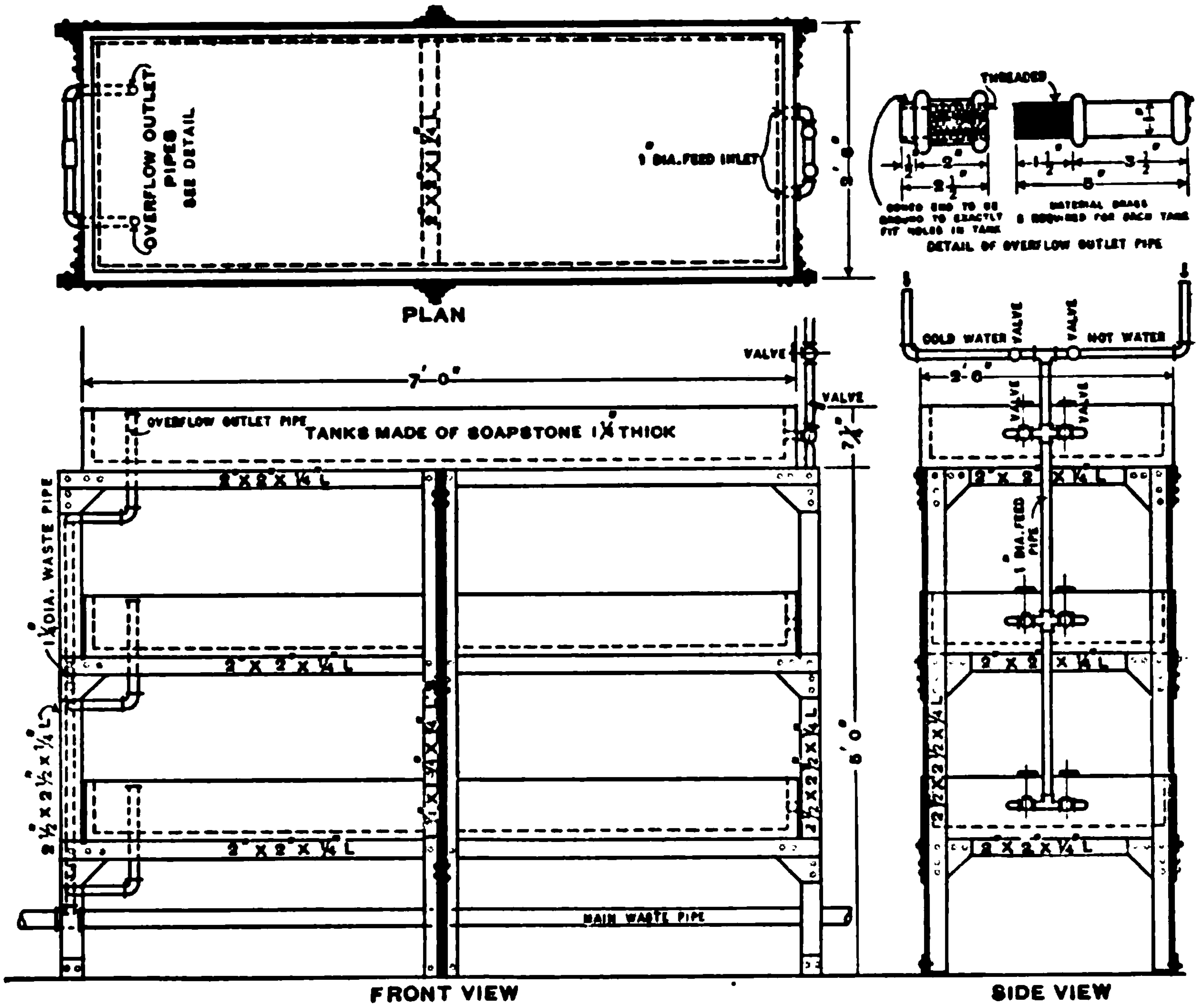

FIG. 16.—Immersion Tanks. (*See p.* 75.)

clip is to be used without cushioning at the points of contact with the test specimen. The bearing at each point of contact should be ¼ in. wide, and the distance between the center of contact on the same clip should be 1¼ in.

68. Test pieces should be broken as soon as they are removed from the water. Care should be observed in centering the briquettes in the testing machine, as cross-strains, produced by improper centering, tend to lower the breaking strength. The load should not be applied too suddenly, as it may produce vibration, the shock from which often breaks the briquette before the ultimate strength is reached. Care must be taken that the clips and the sides of the briquette be clean and free from grains of sand or dirt, which would prevent a good bearing. The load should be applied at the rate of 600 lb. per minute. The average of the briquettes of each sample tested should be taken as the test, excluding any results which are manifestly faulty.

Testing machines and their operation are discussed and illustrated on page 93. The actual tensile strength of neat cement and sand mortar is treated on page 99.

Tests have shown that for the highest and most uniform results briquettes should not be removed from the water until, as specified, just before they are broken.

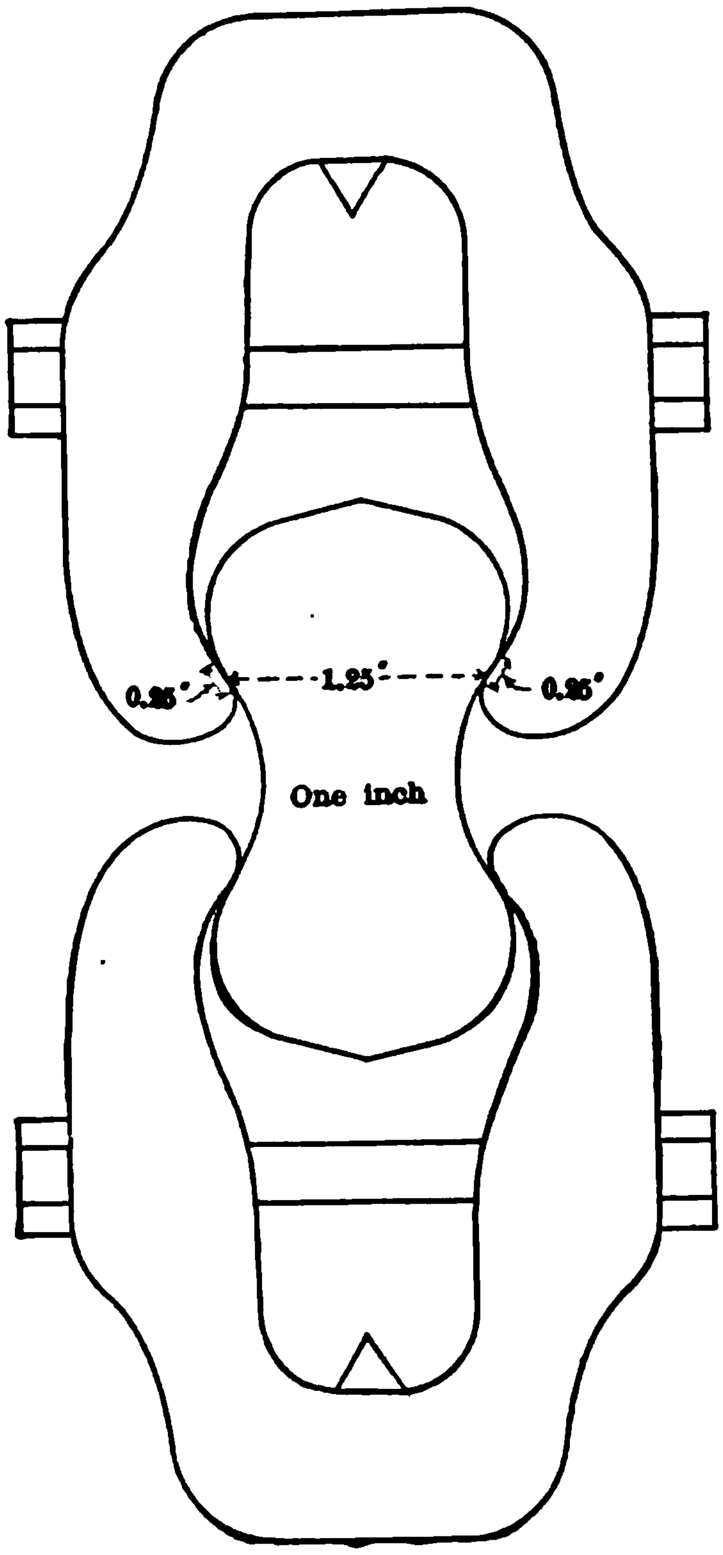

FIG. 17.—Form of Clip. (*See p.* 75.)

Constancy of Volume.* 69. *Significance.* — The object is to develop those qualities which tend to destroy the strength and durability of a cement. As it is highly essential to determine such qualities at once, tests of this character are for the most part made in a very short time, and are known, therefore, as accelerated tests. Failure is revealed by cracking, checking, swelling or disintegration, or all of these phenomena. A cement which remains perfectly sound is said to be of constant volume.

70. *Methods.* — Tests for constancy of volume are divided into two classes: (1) normal tests, or those made in either air or water maintained at about 21° Cent. (70° Fahr.) and (2) accelerated tests, or those made in air, steam or water at a temperature of 45° Cent. (115° Fahr.) and upward. The test pieces should be allowed to remain 24 hours in moist air before immersion in water or steam or preservation in air.

71. For these tests, pats, about 7½ cm. (2.95 in.) in diameter, 1¼ cm. (0.49 in.) thick at the center, and tapering to a thin edge, should be made, upon a clean glass plate [about 10 cm. (3.94 in.) square], from cement paste of normal consistency.

72.—*Normal Test.* — A pat is immersed in water maintained as near 21° Cent. (70° Fahr.) as possible for 28 days, and observed at intervals. A similar pat is maintained in air at ordinary temperature, and observed at intervals.

73. *Accelerated Test.* — A pat is exposed in any convenient way in an atmosphere of steam, above boiling water, in a loosely closed vessel, for 5 hours.

74. To pass these tests satisfactorily, the pats should remain firm and hard, and show no signs of cracking, distortion or disintegration.†

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*Soundness.

†See page 101.

75. Should the pat leave the plate, distortion may be detected best with a straight-edge applied to the surface which was in contact with the plate.

76. In the present state of our knowledge it cannot be said that cement should necessarily be condemned simply for failure to pass the accelerated tests; nor can a cement be considered entirely satisfactory, simply because it has passed these tests.

Submitted on behalf of the Committee,

George S. Webster,
Chairman.
Richard L. Humphrey,
Secretary.

Committee.
George S. Webster,
Richard L. Humphrey,
George F. Swain,
Alfred Noble,
Louis C. Sabin,
S. B. Newberry,
Clifford Richardson,
W. B. W. Howe,
F. H. Lewis.

A steaming apparatus designed by Mr. Richard L. Humphrey is illustrated in Fig. 18.

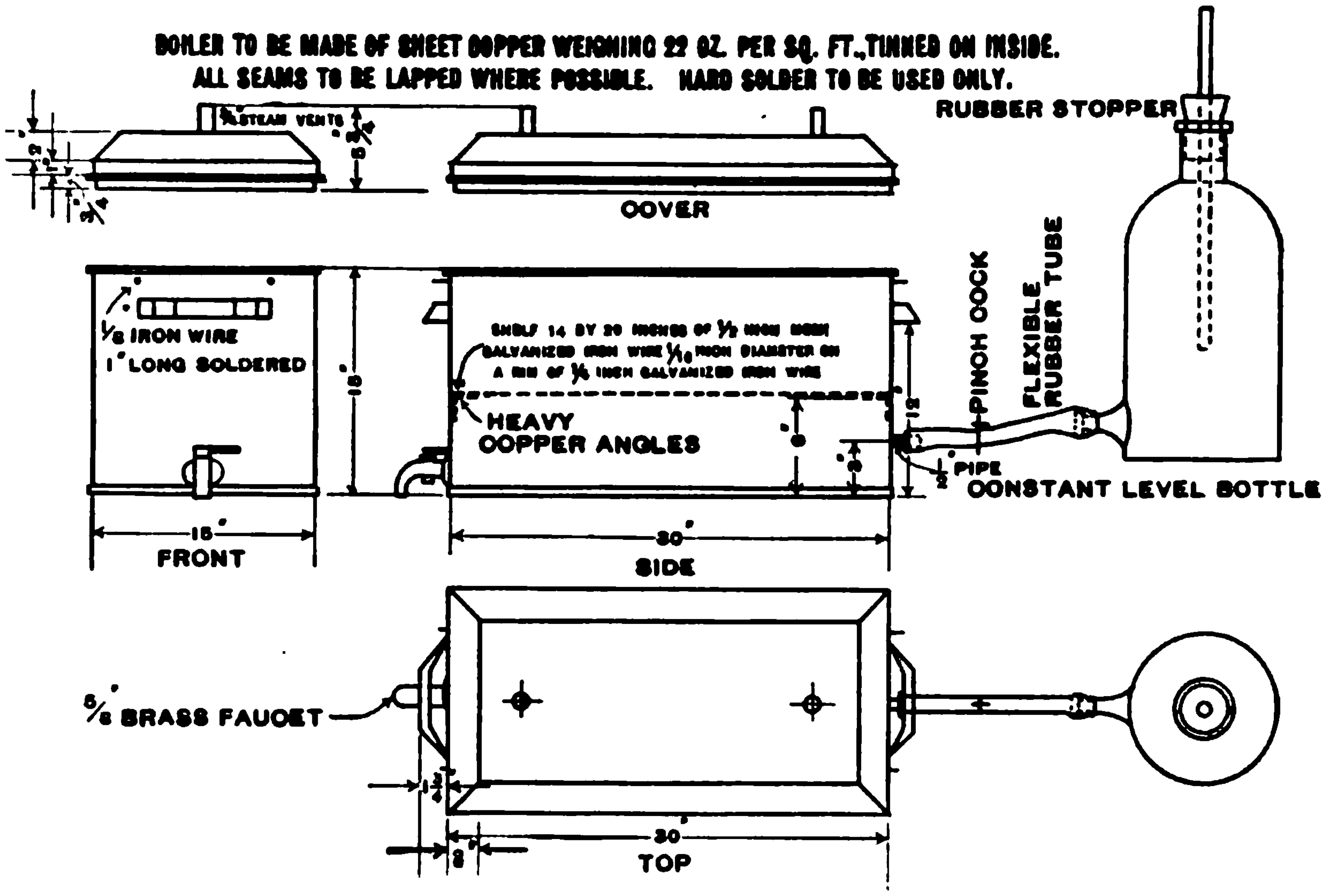

Fig. 18.—Steaming Apparatus. (*See p.* 78.)

ELEMENTARY DIRECTIONS FOR TESTING SOUNDNESS

Soundness tests, which are of greater importance than any other one test, may be made by those unskilled in laboratory practice, with no apparatus except a piece of plate glass at least ½ inch thick and 12 by 18 inches square, pieces of window glass 4 inches square, and a small trowel. Take samples at random from several barrels or bags, as described on page 64. From each sample make three pats of neat cement, requiring for the three about 8 ounces (250 grams) or one cupful of dry cement.

Cements of different classes and degrees of fineness require different percentages of water. The consistency must be such that the cement can be readily kneaded without crumbling and formed into a smooth pat with a thin edge, when pressed upon the piece of glass provided for it, without running or losing its shape.* Approximate amounts may be taken for the first trial of any cement, as, —

Portland Cement....................	20% of water by weight
Natural "	30% " "
Puzzolan "	18% " "

If these quantities after kneading give too wet or too dry a mixture, the paste should be thrown away and the trial repeated with less or more water until the desired consistency is attained. The percentage thus determined may generally be used in the remaining tests of the same shipment of cement.

Place a sample of the dry cement upon the plate glass in the form of a mound, and with the small trowel make a depression in the center. Weigh, or measure, a quantity of water which has been found by trial to give the proper consistency, and pour it into the depression, allowing it to soak into the cement, and then turn the material on the edges into the water with a trowel. As soon as the water is absorbed, the paste is kneaded for 1½ minutes with the hands, which should be protected with rubber gloves.

A piece of window glass about 4 inches square is required for each pat. A portion of the paste is made into a ball and pressed upon one of these pieces of glass so as to form a circular pat about 3 inches in diameter and ½ inch thick in the center, tapering to a thin edge. For the first 24 hours, to prevent the surface from drying too quickly, the pats must be kept under a cloth moistened and suspended above the pats, with its ends immersed in water to keep it wet. The temperature of the air while mixing, and of the water for mixing and storage, should be maintained as near as possible to 70° Fahr. (21° Cent.). At the end of 24 hours one pat should

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*See also Boulogne method, p. 70.

be placed in water and another in air, to be observed at intervals for a period of 28 days, and the third pat placed upon some sort of a frame in a loosely covered vessel over boiling water, and kept there, with the water boiling, for 5 hours. The possible defects which are mentioned above in paragraphs 74 and 75 are described at length on page 103.

APPARATUS FOR A CEMENT TESTING LABORATORY†

(The apparatus is designed for one experimenter. Where the number of pieces is not stated, their number depends upon the quantity of cement to be tested.)

*One piece plate glass, one inch thick, 24 by 24 inches square;
*Two or more gangs of 4 or 5 molds each — A. S. C. E. standard (see Fig. 13, p. 73);
*One metric counter scale recording from 10 grams to 1½ kilograms.
*One No. 100 sieve (96 to 100 meshes to the linear inch) about 20 centimeters (7.87 ins.) in diameter and 6 centimeters (2.36 in.) high, made of woven brass wire cloth, with wires 0.0045 inches diameter;
*One No. 200 sieve (188 to 200 meshes to the linear inch) of similar size to the No. 100 sieve, and made of woven brass wire cloth, with wires 0.0024 inches diameter;
*One measuring glass graduated to 250 cubic centimeters;
*One 8-inch mason's trowel;
*One 4-inch pointing trowel (see Fig. 14, p. 74);
*One-half dozen pairs rubber gloves;
*Pieces of window glass 4 inches square for soundness tests;
*One tensile testing machine (see Figs. 22 to 27, pp. 94 to 98);
*Air thermometer;
*Standard sand;
Two or more gangs of 4 molds each for 2-inch cubes (see Fig. 43, p. 119);
Two or more molds for transverse specimens 1 by 1 by 6 inches (see Fig. 44, p. 121);
10-pound tin cans with covers for holding samples;
One special scale for weighing cement in ascertaining fineness (see Fig. 10, p. 68);
One pan of same diameter as the sieves and 5 centimeters (1.97 in.) deep, with cover, for holding sieve when shaking it;
One measuring glass graduated to 100 cubic centimeters;

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*An asterisk designates the apparatus required for a temporary laboratory on construction work.

†This list has been criticised and approved by Mr. Richard L. Humphrey.

One cement sampler 24 inches long (see Fig. 8, p. 64)
One and one-half minute sand glass;
One moist closet (see Fig. 15, p. 75);
Galvanized iron waste cans;
Apparatus for steaming and boiling specimens (see Fig. 18, p. 78);
Tanks for immersing specimens (see Fig. 16, p. 76);
Vicat needle apparatus (see Fig. 11, p. 69);
One compression testing machine (adapted also to transverse tests), capacity 50,000 lb. (see Figs. 41 and 42, pp. 117 and 118);
Chemical thermometer;
Specific gravity apparatus (see Fig. 9, p. 66);
Microscope with 1½ inch objective;
Set of sieves, about 8-inch diameter, for analyzing sands, sizes No. 10, 15, 20, 30, 40, 60, 74, 150, 200 (the number corresponds to the number of meshes to the linear inch) (see p. 190);
Mechanical shaker for sifting sand (see Fig. 68, p. 189).

SPECIFIC GRAVITY OF DIFFERENT CEMENTS

The specific gravity test, by determining whether a cement is thoroughly burned, supplements the chemical analysis, since the latter does not indicate the degree of calcination. A Puzzolan cement may be distinguished from a true Portland because its specific gravity is usually between 2.7 and 2.9, while that of Portland ranges from 3.01 to 3.15. The adulteration of Portland cement lowers its specific gravity, because the substances used, — powdered sand, limestone, trass or slag, — are lighter than particles of pure cement.

Natural cement usually has a specific gravity above 2.75, ranging from this sometimes as high as 3.2,* thus overlapping the inferior limit given for Portland cement.

The specific gravity of cement is lowered by exposure, because of the absorption of water and carbonic acid, hence the necessity of drying it at 100° Cent. (212° Fahr.) before determining. Even this temperature may not always be sufficient to restore old cements to their original condition.†

A neat little device for dropping fine material into a specific gravity apparatus so as to prevent the entraining of air has been devised by Mr. Thomas H. Wiggin. A thin wooden board with a circular hole in it is

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*Tests of Metals, U. S. A., 1901, p. 476.

†See experiments in Tests of Metals, U. S. A., 1901, p. 476, and Dr. H. Kupfender in *Thonindustriezeitung*, translated in *Cement*, March, 1903, p. 23.

placed above the apparatus and a paper funnel fitted into the hole and filled with dry cement. An electro-magnet, such as is used with an ordinary electric door-bell, is connected with its storage battery and arranged so that the clapper, instead of striking a bell, strikes a metal plate attached to the corner of the board. The constant tapping jars the funnel so that the grains fall slowly into the apparatus without requiring the attention of the operator.

ADVANTAGES OF FINE GRINDING

The effects of fineness of grinding upon cements are to make them,—

- Stronger when tested with sand;
- Weaker when tested neat;
- Quicker setting;
- Capable of producing a larger volume of paste;
- Less affected by free lime.

Fineness is expressed by the percentage of the total weight of the cement retained on each sieve.

A recognition of the value of extreme fineness has led to the adoption of higher standards than formerly, and manufacturers have accordingly improved the quality of their product in this respect. As an illustration of this, in 1875 it was a common requirement for Portland cement that 85% should pass, or not more than 15% be retained on, a sieve having 50 meshes per linear inch; in 1893 Max Gary gave the German standard as 90% to pass, or not more than 10% to be retained on, a sieve having 76 meshes per linear inch, while in 1904 specifications for first-class work required not more than from 6% to 10% to be retained on a sieve having 100 meshes per linear inch, and not more than 20% to 35% on a sieve having 200 meshes per linear inch. Some American factories are equipped to grind even finer than this, shipping cement of which less than 10% is retained on a No. 200 sieve. Standard requirements for different cements are given in the specifications on pages 30 and 31.

Strength affected by Fineness. With the same proportions of sand higher tensile and compressive strength is obtained from finely ground than coarsely ground cements. Conversely, a larger proportion of sand can be used with fine ground than with coarse ground cement, with the same resulting strength.

The chief cementing value of a cement lies in the grains which are fine enough to pass a sieve having 200 meshes per linear inch. Photographs of thin sections of sand briquettes several years old made by

Mr. E. W. Lazell show very clearly the coarser grains of cement which have never been penetrated and chemically changed by the water.

Tested neat, a coarse cement may give higher strength than the same cement after regrinding. This is chiefly due, in the opinion of the authors, to the fact that the fine cement requires more water in gaging to produce the same consistency of paste, so that the same weight of cement produces a larger volume of paste, which therefore has less density and consequently lower strength. When sand is added, on the other hand, less influence is exerted by the water, because in any case a smaller volume of it is required in proportion to the dry materials, and besides this the very fine grains, which also have higher cementing qualities, fit more readily into the voids in the sand. The relation of the density of a mortar to its strength is discussed in Chapter IX, page 132.

The effect of the fineness of cement upon its strength was brought to general notice by Mr. John Grant[*] in 1880, who quotes experiments made in Germany by Dyckerhoff. In 1883 Mr. I. J. Mann[†] illustrated the small cementing value of the coarse particles by substituting for them grains of sand of the same size, with but little reduction in the resulting strength.

The following table from tests reported in 1885 by Mr. Eliot C. Clarke[‡] illustrates the effect of the fineness of cement on paste and mortars. All of these cements would be reckoned as coarse in modern practice, but the relative results are still of interest.

Tensile Strength of Mortar Affected by Fineness of Cement.

BY ELIOT C. CLARKE.

PORTLAND CEMENT			ROSENDALE CEMENT		
Proportions of cement to sand	STRENGTH IN POUNDS PER SQUARE INCH		Proportions of cement to sand	STRENGTH IN POUNDS PER SQUARE INCH	
	Ordinary cement 35% retained on No. 120 sieve	Finely ground cement 12% retained on No. 120 sieve		Coarse cement 17% retained on No. 50 sieve	Fine cement 6% retained on No 50 sieve
1:0	403	304	1:0	98	92
1:3	105	180	1:1½	29	41
1:5	68	96	1:2	16	25

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[*] Proceedings Institution of Civil Engineers, Vol. LXII, p. 149.

[†] Proceedings Institution of Civil Engineers, Vol. LXXI, p. 254.

[‡] Transactions American Society of Civil Engineers, Vol. XIV, p. 158.

Mr. D. B. Butler* in England has made extended tests to determine the value of coarse particles in cement and the effect of regrinding. A summary of one of his tables, illustrating also the effect of fineness upon the time of set, gives the average of his results from four brands of Portland cement.

Effect of Regrinding Coarse Particles and of Substituting Sand.

BY DAVID B. BUTLER.

CEMENT, HOW TREATED	Fineness residue per cent on sieves of meshes per linear inch			Setting Properties		TENSILE STRENGTH IN POUNDS PER SQUARE INCH									
						Neat cement					1 part cement to 3 parts sand				
	180	76	50	Initial set min.	Final set min.	7 days	28 days	3 mo.	6 mo.	12 mo.	7 days	28 days	3 mo.	6 mo.	12 mo.
As received	33.7	15.5	4.6	13	90	504	580	641	702	717	194	262	354	404	421
Reground	1.3	0.0	0.0	2	20	497	478	518	489	504	326	411	531	591	618
Sand substituted for coarse particles†.						414	480	606	660	702	164	217	290	354	387

The fine grinding of commercial cements, by accelerating the setting, has been one of the causes for the necessity of adding gypsum or plaster during manufacture.

American vs. European Sieves. Standard sieves recommended by the American Society of Civil Engineers‡ and the French Commission§ are tabulated below with English and Metric equivalents.

American Sieves.

U. S STANDARD					METRIC EQUIVALENTS			
No. of sieve	Meshes per linear inch	Meshes per square inch	Diam. of wire in.	Width of openings in.	Meshes per cm.	Meshes per sq. cm.	Diam. of wire mm.	Width of openings mm.
100	100	10 000	0.0045	0.0045	39	1 550	0.114	0.140
200	200	40 000	0.0024	0.0026	79	6 200	0.061	0.066

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*Proceedings Institution of Civil Engineers, Vol. CXXXII, p. 343, and Butler's Portland Cement, 1899, p. 169.

†All particles not passing No. 180 sieve (averaging 33.7% by weight) were removed from the original cement as received, and sand having grains of similar size substituted for them.

‡See p. 67.

§Commission des Méthodes d'Essai des Matériaux de Construction, 1894, Vol. I, p. 248.

French Sieves.

FRENCH STANDARD				ENGLISH EQUIVALENTS			
Meshes per cm.	Meshes per sq. cm.	Diam. of wire mm.	Width of openings mm.	Meshes per linear inch	Meshes per square inch	Diam. of wire in.	Width of openings in.
18	324	0.20	0.36	46	2 120	0.0078	0.0124
30	900	0.15	0.18	76	5 780	0.0059	0.0071
70	4 900	0.05	0.09	178	31 680	0.0020	0.0035

Separating Material Passing No. 200 Mesh Sieve. The high cementing value of the grains of cement passing a No. 200 sieve leads in elaborate tests to still finer separations. In studies for soil analysis chiefly, the various methods of separating the different sized grains have been developed. They are fully described in Wiley's *Principles and Practice of Agricultural Analysis*, Vol. I, pages 171 to 281. The same principles are applicable to cement determinations, except that some liquid other than water must be employed.

Separation may be made by a winnowing device* in which a blast of air is directed against falling grains of cement; by settlement through water at rest, which in its simplest form may be accomplished by allowing the material to settle in a beaker, for a certain length of time and then decanting†; and by means of a liquid in motion, as illustrated in the Schöne apparatus, and, with still greater exactness, by Hilgard's churn elutriator.‡ The Schöne apparatus has been adapted by Dr. W. Michaelis to cement, and has also been employed by Mr. J. B. Johnson.§

QUANTITY OF WATER FOR NEAT PASTE AND MORTAR

The quantity of water used in gaging affects the results of tests, especially in the determination of the time of setting and of the strength. (See p. 151.) Different cements even of the same class require different proportions of water to produce the same consistency, chiefly because of differing degrees of fineness, the cement containing the largest proportion of fine particles requiring the largest percentage of water by weight.

Percentage of Water for Mortar of Normal Consistency. The fol-

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*Tests of Metals, U. S. A., 1901, p. 474.

†Allen Hazen in Report Massachusetts State Board of Health, 1892.

‡Wiley's Principles and Practice of Agricultural Analyses, 1894, Vol. I, p. 226.

§Johnson's Materials of Construction, 1903, p. 412.

lowing table, based on the formula of Mr. Feret given on page 88, which is strictly applicable only to French sands and French methods, has been suggested provisionally by the Committee of the American Society for Testing Materials (1904), for the percentage of water for mortars of consistency corresponding to that of normal neat paste. To use the table select from the first column the percentage of water required for the neat paste of the selected cement and read in column of the desired proportions the percentage of water required for the mortar in terms of the sum of the weights of the cement and sand.

Percentage of Water for Cement Mortars of Normal Consistency.

Percentage of water for neat cement	PERCENTAGE OF WATER FOR SAND MORTARS					ntage of water neat cement	PERCENTAGE OF WATER FOR SAND MORTARS				
	Proportions cement to sand by weight						Proportions cement to sand by weight				
18	12.0	10.0	9.0	8.4	8.0	33	17.0	13.3	11.5	10.4	9.6
19	12.3	10.2	9.2	8.5	8.1	34	17.3	13.6	11.7	10.5	9.7
20	12.7	10.4	9.3	8.7	8.2	35	17.7	13.8	11.8	10.7	9.9
21	13.0	10.7	9.5	8.8	8.3	36	18.0	14.0	12.0	10.8	10.0
22	13.3	10.9	9.7	8.9	8.4	37	18.3	14.2	12.2	10.9	10.1
23	13.7	11.1	9.8	9.1	8.5	38	18.7	14.4	12.3	11.1	10.2
24	14.0	11.3	10.0	9.2	8.6	39	19.0	14.7	12.5	11.2	10.3
25	14.3	11.6	10.2	9.3	8.8	40	19.3	14.9	12.7	11.3	10.4
26	14.7	11.8	10.3	9.5	8.9	41	19.7	15.1	12.8	11.5	10.5
27	15.0	12.0	10.5	9.6	9.0	42	20.0	15.3	13.0	11.6	10.6
28	15.3	12.2	10.7	9.7	9.1	43	20.3	15.6	13.2	11.7	10.7
29	15.7	12.5	10.8	9.9	9.2	44	20.7	15.8	13.3	11.9	10.8
30	16.0	12.7	11.0	10.0	9.3	45	21.0	16.0	13.5	12.0	11.0
31	16.3	12.9	11.2	10.1	9.4	46	21.3	16.1	13.7	12.1	11.1
32	16.7	13.1	11.3	10.3	9.5						

Weights of Cement and Sand for Different Proportions.

	1:1	1:2	1:3	1:4	1:5
Cement	500	333	250	200	167
Sand.............	500	666	750	800	833

The Engineers of the U. S. Army* advocate a dryer mixture than most

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*Professional Papers, No. 28.

authorities, and the following percentages suggested by them may therefore be taken as representing minimum quantities.

Portland Cement.

Neat	20% of water by weight.
1 cement: 3 sand	12½% " "

Natural Cement.

Neat	30% of water by weight.
1 cement: 1 sand	17% " "

Puzzolan Cement.

Neat	18% of water by weight.
1 cement: 3 sand	10% " "

French Determination of Consistency of Neat Paste. The Vicat needle apparatus has been selected in America as well as in France as the standard appliance for determining normal consistency. The apparatus is shown in Fig. 11 on page 69, and the U. S. standard method of applying the test is there described.

A plastic paste is preferred to one of dryer consistency. The French Commission* advised a softer consistency than the American standard, the French requiring for normal consistency the penetration of a needle one centimeter (0.39 in.) in diameter and weighing 300 grams (10.58 oz.) through a disc of cement 40 millimeters (1.57 in.) thick to within 6 millimeters (0.23 in.) of the bottom, making a total depth of penetration of 34 millimeters (1.33 in.), while the American Society recommend the penetration of a similar needle into a like mass to a depth of 10 millimeters (0.39 in.) below the surface.

Feret's Formula† for percentage of water for mortar of normal consistency was evolved from a very interesting series of experiments.‡ He found that it was impracticable to determine with the Vicat needle the proper consistency of a mortar of cement and sand, and therefore based his determination upon the average judgment of several operators, plotting the consistencies designated by them upon cross-section paper.

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*Commission des Méthodes d'Essai des Matériaux de Construction, 1894, Vol. I, p. 270.

†Commission des Méthodes d'Essai des Matériaux, 1895, Vol. IV, p. 103.

‡Methods of Mr. Feret's investigations are described and illustrated in an article by the authors on "Quantity of Water to Use in Gaging Mortars" in *Cement and Engineering News* (Chicago), November, 1903.

His formula is:

For mortars of plastic consistency,[*]

$$W = \frac{2}{3}\,\frac{P}{S+1} + 6.0 \qquad (1)$$

For mortars of dry consistency,[*]

$$W' = \frac{2}{3}\,\frac{P}{S+1} + 4.5 \qquad (2)$$

Where

W = percentage of water for mortar in terms of weight of the mixture of dry materials;

P = percentage of water required for neat cement of normal consistency;

S = parts of sand by weight to one part cement.

Mr. Richard L. Humphrey[†] states that from formula (2) he has obtained very uniform results with U. S. standard sand, although slight modifications are necessary for a mortar containing more or less than three parts of sand.

ARBITRARY PERIODS OF SETTING

The methods employed in mixing and depositing the mortar or concrete and the character of the construction form a guide to the necessary requirements for the time of setting of the cement.

The setting of cement is due to chemical reaction, as described by Mr. Spencer B. Newberry on page 57. The process is a gradual one, but may be arbitrarily divided into three periods:

Initial set.
Final set.
Hardening.

The dividing line between these periods is arbitrary, but the division is based upon the fact that after water is added the paste remains plastic for a certain period, and then commences to "stiffen" or crystallize. This is called the time of initial set. The setting process continues rapidly, and when a point is reached that the paste will withstand a certain pressure, arbitrarily fixed in practice, it is said to have reached its final set. The

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[*] The original formula of Mr. Feret corresponding to formula (2) is $E = \frac{2}{3} NA + 60$, and to formula (3) is $E = \frac{2}{3} NA + 45$, in which E = weight of water in grams required for one kilogram of dry mixture of cement and sand, N = weight of water in grams required for one kilogram of neat cement, and A = weight in kilograms of cement in one kilogram of the dry mixture. The change in the form of the formula permits the direct use of percentages.

[†] Journal Franklin Institute, 1901–2.

process of hardening now continues more slowly, and proceeds with increasing slowness for an indefinite period.

Those unfamiliar with cement construction must bear in mind that a cement which has reached its final "set" is not hard nor is it capable of bearing a load. Natural cement, for example, usually reaches its initial and its final set much earlier than Portland cement, but it hardens more slowly, and Natural cement masonry will not bear loading nearly so quickly as Portland cement masonry.

EUROPEAN METHODS FOR DETERMINING SET

The French and German requirements are similar to the American (p. 70) except that in them the commencement of the set is taken as the time when the needle can no longer penetrate entirely to the bottom of the box instead of limiting it to a penetration of 5 millimeters below the surface.

For sand mortars the French Commission designate the final set as the moment when the surface of the mortar can support pressure of the thumb without indentation. As an alternate method, they use the Vicat apparatus with a needle one centimeter (0.39 in.) in diameter and weighing 5 kilograms (11.02 lb.). The preliminary reports of Mr. R. Feret and Mr. P. Alexander in Commission des Méthodes d'Essai des Matériaux de Construction, 1895, Vol. IV, pp. 111 and 139, describe experiments with different apparatus.

Comparison of Vicat and Gillmore Needles. The Gillmore needles, the former American standard, were first used by General Totten in 1830.*

By these needles the initial set of Neat cement is the time at which a wire one-twelfth-inch diameter, loaded to a ¼ pound, is just supported by the mass without appreciable indentation. The final set is taken as the time when a wire one-twenty-fourth-inch diameter, loaded to weigh one pound, is supported without appreciable indentation.

The diagram in Fig. 19, page 90, from experiments made at the Watertown Arsenal† upon various cements (designated by letters) shows the difference in the nominal time of setting when measured by the Gillmore needle and the Vicat needle, employing with the latter the German method. (See above.) The diagram also shows the variation in time of set of Portland cement occasioned by varying the proportion of water, and the effect of leaving out the usual "restrainer" of plaster of Paris or gypsum.

*Gillmore's Treatise on Limes, Hydraulic Cements and Mortars, p. 80.

†Tests of metals, U. S. A., 1901, p. 492.

THE RATE OF SETTING

The rate of setting of cement, that is, the process of hardening, has been studied by the French Commission* in France and by Prof. Edgar B. Kay in the United States. The diagram, Fig. 20, page 91, shows curves of setting made with a machine of Prof. Kay's design and the corresponding tensile strength of briquettes of the same cement. Prof. Kay calls attention to the positive change from the plastic to the granular or crystalline structure which in the cement shown occurred between the periods of 35 and 40 minutes. The elongation of the briquette when being broken gradually changed from ½ inch at the 5-minutes period to 0.15 inch at 40 minutes, while at 200 minutes, or one hour before the initial set was completed, the elongation was not measurable.

FIG. 19.—Time of Setting of Typical Cements,—and comparison of Vicat and Gillmore's Needles. (Tests of Metals, U. S. A., 1901.) (*See p.* 89.)

AMERICAN AND EUROPEAN STANDARD SANDS COMPARED

The character of the sand has so great an effect upon the strength of a mortar that for comparing different brands of cement or specifying requirements of strength a sand of standard size and quality is essential.

*Commission des Méthodes d'Essai des Matériaux de Construction, 1895, Vol. IV., p. 111.

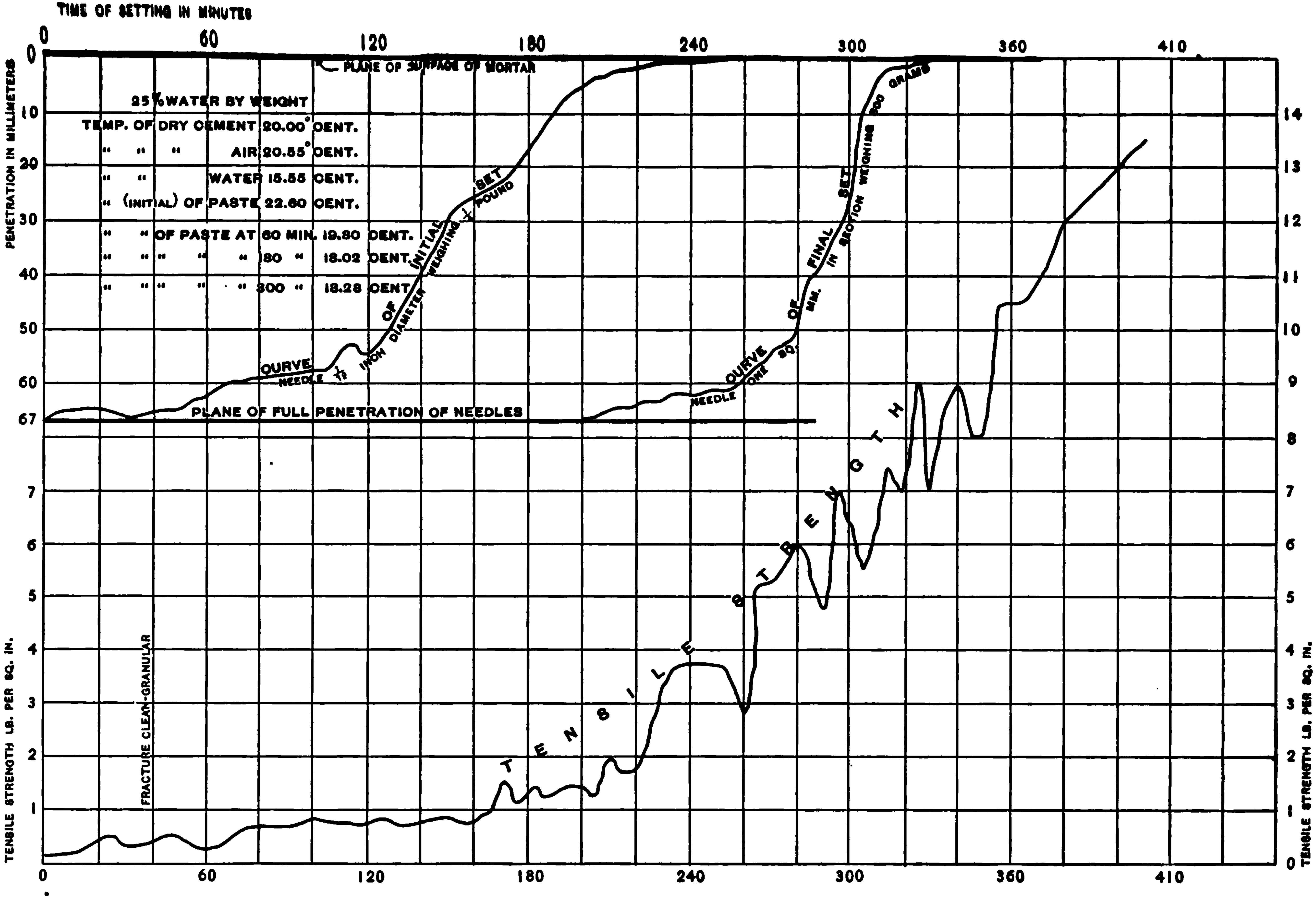

FIG. 20.—Rate of Setting and Corresponding Tensile Strength of Portland Cement Paste. (*See p.* 89.)
(Especially prepared by Prof. Edward B. Kay for this treatise.)

The U. S. Standard Sand recommended by the Committee of the American Society of Civil Engineers, as specified on page 71, is a natural sand from Ottawa, Ill., screened to pass a sieve having 20 meshes per linear inch, and retained on a sieve having 30 meshes per linear inch.

The change in America from artificial to natural sand is in accord with recent practice abroad.

The English Standard Sand is obtained from a pit at Leighton Buzzard,* and the screens are the same as in the United States.

The German Standard Sand is a natural quartz retained between sieves having respectively 20 and 28 meshes per linear inch.

The French Standard Sand, a natural sand from Leucate, France, is simple or compound. Simple standard sand must pass a screen having holes 1.5 millimeters (0.059 in.) in diameter, and be retained on a screen having holes one millimeter (0.039 in.) in diameter. Compound standard sand is made by forming a mixture of equal weights of the following:

(1) Grains passing holes of 2 mm. (0.079 in.) and retained by 1.5 mm. (0.059 in.).
(2) Grains passing holes of 1.5 mm. (0.059 in.) and retained by 1 mm. (0.039 in.).
(3) Grains passing holes of 1 mm. (0.039 in.) and retained by 0.5 mm. (0.020 in.).

THE FORM OF BRIQUETTE FOR TENSILE TESTS

Mr. John Grant in 1871† presented results of a series of experiments with different forms of briquettes and sizes of section. Ten years later‡ he adopted the form now used in England which is substantially the same as that recommended by the American Society of Civil Engineers in 1884, and, with a very slight alteration, in 1903. (See Fig. 12, p. 72.)

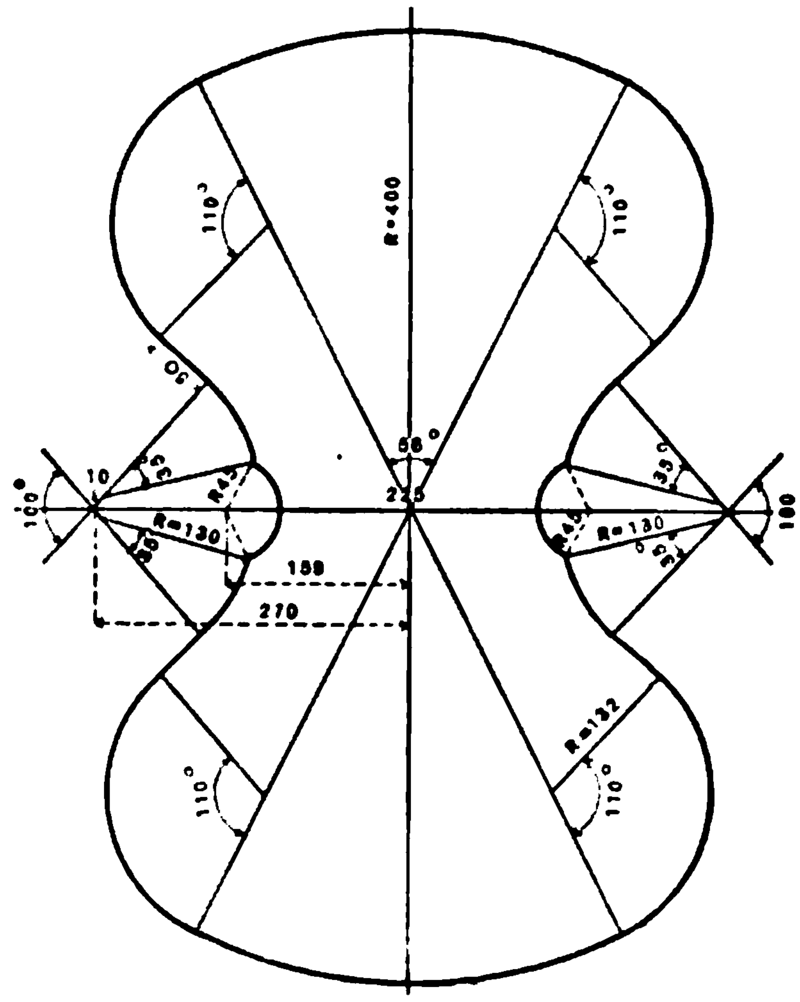

FIG. 21.—The German Standard Briquette (dimensions are in millimeters). (*See page* 92.)

The German Standard Briquette, also adopted by the French Commission

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*Butler's Portland Cement, 1899, p. 200.

†Proceedings Institution of Civil Engineers, Vol. XXXII, p. 282.

‡Proceedings Institution of Civil Engineers, Vol. LXII, p. 137.

in 1893, is shown in Fig. 21. The section is 5 square centimeters (0.78 sq. in.). Results with this form of briquette are lower per unit of area than those of the American pattern. Prof. Jerome Sondericker* in studying the quality of strength and uniformity of breaking of different forms, found that a groove in the sides of the specimen lowered the unit strength about 13%.

M. Feret† found that briquettes of 5 square centimeter section gave 46% higher strength per unit of area than briquettes of 16 square centimeter, and attributed this difference to lack of homogeneity throughout the section.

TO CONVERT METRIC UNITS OF STRENGTH TO ENGLISH UNITS

To convert values of kilograms per square centimeter (kg. per sq. cm.) to pounds per square inch (lb. per sq. in.), multiply the former by 14.2.‡ To convert values of pounds per square inch (lb. per sq. in.) to kilograms per square centimeter (kg. per sq. cm.), multiply the former by 0.07.§

MACHINES FOR TESTING TENSILE STRENGTH

A testing machine should be so designed that the strain can be applied to the briquette at a definite rate without irregularity or jar. The clips should be suspended from pivoted bearings to avoid friction, and should be stiff, so that they will not spread. The contact surfaces should hold the briquette firmly without crushing it.

Effect of Eccentricity in Placing Briquettes. One of the causes of irregularity in tests of similar briquettes is careless adjustment of the briquette in the clips of the machine, that is, placing it so that it is not exactly central. Prof. J. B. Johnson‖ has discussed this theoretically, and concludes that

if h = width of specimen,
and a = eccentricity of loading,
then $\frac{6a}{h}$ represents the percentage of increase in stress due to eccentricity.

"Thus if a cement briquette one inch thick be placed in the clips 0.01 inch out of center, its strength will be reduced by 6%. This assumes perfect freedom of motion of the clips at the surface of contact, which they do not

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*Journal Association of Engineering Societies, January, 1899, p. 1.

†See p. 136.

‡More exactly, 14.2234.

§More exactly 0.07031.

‖1903 Edition, p. 446.

have. Experiments made at the Massachusetts Institute of Technology have shown that a displacement of one-sixteenth inch decreased the tensile strength by from 15% to 20%."

Rate of Applying Strain. The selections of the standard rate of 600 lb. per minute by the committee of the American Society of Civil Engineers (see p. 76) is based on an extensive series of tests from which it was found that the breaking load increases with the speed up to a rate of at least 800 lb. per min., but that between the rates of 400 and 600 lb. the variation is slight. Mr. E. S. Wheeler's* experiments tend to confirm this conclusion.

FIG. 22.—Shot Testing Machine. (*See p.* 95.)

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*Report Chief of Engineers, U. S. A., 1895, p. 2916.

Types of Testing Machines. There are three most common types of tensile testing machines.

(*a*) The shot machine, originally designed by Dr. Michaelis and shown in its American patterns in Figs. 22 and 23, applies the load by the discharging of a stream of shot whose flow is automatically shut off when the break occurs. The breaking load is determined from the weight of the shot.

FIG. 23.—Shot Testing Machine. (*See p.* 95.)

(*b*) The simple or compound lever machines apply their load by a sliding weight operated by hand or by power. A compound lever power machine is illustrated in Fig. 24, page 96.

(c) The spring balance machine, which was originally designed and used by Mr. Henry Faija in England, transmits the strain from the crank to the briquette through a spring balance which records the load upon the dial. (See Fig. 25, p. 97.)

Johnson's Ring Testing Machine. A machine devised by Mr. A. N. Johnson for testing the tensile strength of cement and mortars is based on an entirely different principle from the clip machines just described.

FIG. 24.—Compound Lever Testing Machine (*See p.* 95.)

The cement or mortar instead of being formed into standard briquettes is molded in the shape of rings. The apparatus is shown in Figs. 26 and 27, page 98. A cylinder A filled with water or other liquid contains a piston operated by a handwheel F. The pressure exerted by lowering the piston is transmitted by the liquid to the closed cylinder B, a section of which consists of rubber tubing which is expanded by the pressure from within until it bursts the ring of cement which encircles it. The pressure is also

transmitted to the gage whose reading for a certain diameter and thickness of ring of cement or mortar bears a definite ratio to the circumferential tensile stress upon the ring. Brass molds of special design for forming the rings are constructed either single or in gangs of five.

TENSILE TESTS OF NEAT CEMENT AND MORTAR

Tests of tensile strength are made primarily to determine whether the ingredients of the cement and the process of its manufacture are such that a continued and uniform hardening may be expected in the work, and whether its actual strength in mortar or concrete is so high that it can be depended upon to withstand the strain placed upon it. Tensile tests must be combined with other tests, most particularly the test for soundness, to arrive at correct conclusions on these points.

The dates which have been universally selected for making tensile tests to determine the quality of the cement are 7 days and 28 days after molding. In each case the briquettes remain for the first 24 hours in moist air, and the balance of the time in water at the standard temperature of 21° Cent. (70° Fahr.). For arriving at a quicker knowledge of the quality, standard specifications require one-day tests, the briquettes being broken after 24 hours in moist air. Longer periods than 28 days are useful for determining the rate of permanent hardening, although the rate of growth is different in neat cements, mortars and concretes. The growth in tensile strength is not strictly comparable with its growth in compressive strength.

FIG. 25.—Spring Balance Testing Machine. (*See p.* 96.)

A cement giving an extremely high test at a very short period may be regarded with suspicion, although if future tests show a good increase, no fault can be found. Specifications occasionally limit the strength of the one-day or the 7-days test. Others require a definite increase in strength between periods. The engineers of the New York Rapid Transit Com-

FIG. 26.—Machine with Cement Ring in Position ready for a Test. (*See p.* 96.)

FIG. 27.—Machine after a Test with the Top Cap Removed, Showing the Broken Cement Ring and Distended Rubber Cylinder or Tube. (*See p.* 96.)

mission require, for example, "a specific ratio of increase," 15% in tensile strength "from 7 to 28 days, and furthermore that a cement showing as high as 750 lb. at the earlier stage should be generally refused as unlikely to give good results in long-time tests."* Manufacturers consider this a very severe requirement for Portland cement tested neat.

Specifications for tensile strength are given on pages 30 and 31. A comparison of these with the actual strengths of different cements as furnished by manufacturers will show that on the average the tensile strength of Portland cement as now manufactured is largely in excess of the specifications. In comparing these figures, however, it must be recognized that specifications are not for average strength, but are intended to cover the lowest limit which can be allowed on the work, and to provide for lack of uniformity in testing as well as in real quality.

GROWTH IN STRENGTH OF PORTLAND AND NATURAL CEMENTS AND CEMENT MORTARS

The curves in Fig. 28, for which we are indebted to Mr. W. Purves Taylor, illustrate the growth in strength of neat Portland cement and Portland cement mortars. The tests from which the curves are drawn were made under his direction at the Philadelphia Municipal Laboratories.

FIG. 28.—Growth in Tensile Strength of Neat Portland Cement and Portland Cement Mortars with Different Proportions of Standard Sand. (*See p.* 99.)
(Compiled for this treatise by W. Purves Taylor.)

*Report of New York Board of Rapid Transit Commissioners, 1900-01, p. 258.

The neat and 1:3 (*i. e.*, one part cement to 3 parts sand by weight) curves are averaged from over 100,000 briquettes, while the other curves are each based on tests of 300 to 500 briquettes.

The cements included a number of brands, American brands largely predominating. The sand was crushed quartz, the former U. S. standard. The Philadelphia records include tests of much longer time than one year, and there is a noticeable falling off in the observed tensile strength after the one-year period. This is most noticeable with neat cement of rotary kiln brands, but also occurs to a less degree with sand mortars. With cements from stationary kilns it is less marked. The falling off in tensile tests is generally attributed to the brittleness of the small sized specimens, which tends to irregularity of results with the ordinary testing machine, and to the unequal hardening of the surface and interior of the specimen, rather than to actual deterioration in the cement.

The average growth in strength of neat Natural cement and Natural cement mortars is illustrated in Fig. 29 from data kindly prepared by

FIG. 29.—Growth in Tensile Strength of Neat Natural Cement and Natural Cement Mortars with Different Proportions of Standard Sand. (*See p.* 100.)
(From data by Richard L. Humphrey and A. W. Munsell.)

Mr. Richard L. Humphrey from Philadelphia tests, and by Mr. A. W. Munsell from tests made for the Baltimore & Ohio R. R. Cements from seven different sections of the United States are included in the averages from which the curves are drawn, representing the Akron, Cumberland, James River, Lehigh Valley, Louisville, Milwaukee, Rosendale and Utica districts.

SOUNDNESS OR CONSTANCY OF VOLUME

The term "soundness" is more commonly used in America and England than the expression "constancy of volume" suggested by the Committee of the American Society of Civil Engineers, or "deformation" as employed in France. The purpose of the test is to determine in advance whether a cement is in danger of disintegrating, that is, crumbling, or of expanding or contracting so as to cause distortion or cracking in the masonry.

If a cement satisfactorily passes the tests for soundness, it will in all probability withstand the effect of the elements without swelling or disintegration, and will continue to harden for an indefinite period. Failure, on the other hand, to pass the tests for soundness, especially the hot test, is not positive proof of inferiority, for a cement which fails to pass may possibly, through subsequent exposure to the air before being used, or because of mixing with sand or other aggregate, produce durable masonry. We may, however, with safety adopt the following conclusion:

If a Portland cement passes the hot test it may be used immediately with reasonable certainty of its ultimate soundness. If it fails to pass, it should be regarded with suspicion and thoroughly tested.

Causes of Unsoundness. Disintegration, or crumbling, of work in Portland cement properly mixed and laid, is usually due to an excess of lime in a form which can be attacked by the elements. This may come about in two entirely distinct ways, either (1) by the use of too high a proportion of lime in the raw materials from which the cement is made, (2) by under-burning the cement, or (3) by too coarse grinding.

The presence of magnesia in excess in a thoroughly burned cement may produce a gradual expansion which will disintegrate the mortar or concrete after several years. This action, brought to notice by tests of Mr. H. Le Chatelier,* is generally recognized, but opinions differ as to the limit to the percentage of magnesia which may occur in Portland cement without deleterious effect. Le Chatelier's experiments led him to consider 5% as injurious. The Association of German Cement Manufacturers first placed the limit at 3½%, and later raised it to 5%. Mr. Spencer B. Newberry states (page 56) that recent experiments made by himself and by Van Blaese show that cements containing 8% or 9% of magnesia will pass the boiling test, while those with 15% magnesia will expand. The limit of 4% recommended by the Committee of the American Society for Testing Materials in 1904 (see p. 30) is undoubtedly conservative. Natural cement, which is burned at a lower temperature,

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*Commission des Méthodes d'Essai des Matériaux de Construction, 1895, Vol. IV, p. 229.

may contain a much larger quantity of free lime and of magnesia without injury.

The expansion caused by an excess of free lime is due to the hydration or slaking of the calcium oxide (CaO). This is readily understood from the expansion of common lime, which in slaking with water will produce a bulk of paste from 2 to 3 times greater than the volume of the loose powder. The presence of lime in a free or loosely combined state must not be confounded with other compounds of calcium. A thoroughly slaked lime paste, or powder, that is, one which is completely hydrated, may in fact be added to a Portland cement mortar without injurious results, to lengthen its time of setting or to produce a more water-tight mixture.

The small amount of free lime which frequently occurs and sometimes produces unsoundness in first-class Portland cement, tested when fresh, may be hydrated and rendered harmless by air-slaking after, say, two or three weeks' storage, or after spreading the cement out in the air.

Adulteration with slag may cause a cement containing an excess of free lime to pass the boiling test.

Tests for Soundness. The presence of ingredients which will render a cement unsound, that is, which will cause it to expand or disintegrate, is determined by the eye, or by measuring appliances in specimens which have been exposed under conditions which as nearly as possible produce the same effect as the practical effects of time and the elements.

There is apparently no reliable method for determining the presence of free lime by chemical analysis. Mr. E. Candlot* says that "there is in fact no method for finding the percentage of free lime in the cement," and Dr. Schuman* concurs in this view in the following statement·

> I do not know a method for finding out the percentage of free lime in Portland cement. I do not think there exists such a method, and I am myself of the opinion that chemists will never find out one; the solutions capable of taking away the free lime from the cement will always work in a more or less strong degree on the cement itself.

This inability to detect free lime by chemical analysis necessitates a resort to physical tests. Specimens for testing soundness are generally circular pats tapering toward the edges, so that the expansion of the mass will tend to enlarge the circumference and thus produce cracks at the edges.

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*Quoted by W. W. Maclay in Transactions American Society of Civil Engineers, Vol. XXVII, p. 448.

Egg-shaped specimens and also briquettes are sometimes used. Both of these show deterioration by the appearance of the surface.

Appearance of Soundness Specimens. Cracks which appear on pats are not always caused by unsoundness. Expansion cracks, which reveal an unsound cement, are distinguished from shrinkage cracks, which usually appear during setting instead of after the cement is set, in Figs. 30 to 37. Hair cracks also sometimes appear upon specimens, and in practice upon neat cement or very rich mortar, where so large an excess of water has been employed in mixing that it does not dry off until the cement has set, and causes the deposition of a very thin coating of partially decomposed cement which had remained in suspension in the water. An unsound cement in air or in water at the ordinary temperature will generally show defect within 28 days, although in very exceptional cases several months or even years have been known to elapse before signs of deterioration appear in specimens which have not been subjected to heat.

Photographs of pats illustrating the appearance of defective specimens which have been subject to the boiling test are shown in Figs. 38 and 39, pages 108 and 109. Figs. 30 to 37, pages 104 and 105, are sketches* employed in the Philadelphia Municipal Laboratories for distinguishing harmless appearances in neat pats from evidences of unsoundness. Mr. Taylor describes the pats as follows:

Fig. 30 represents a normal pat in good condition.

Fig. 31 represents shrinkage cracks. These cracks are ordinarily due to the use of a too wet mixture or to too quick a drying out. If the pats are left exposed to dry air during setting these cracks are often developed. Shrinkage cracks ordinarily, therefore, indicate only a lack of care in manipulation, and not dangerous properties in the cement.

Fig. 32 shows cracks caused by the expansion of the cement and the curling of the edges of the pat from the glass while the pat still adheres, which is often coincident with the expansion. In the air pats these cracks are developed in nine-tenths of the pats adhering to the glass, and unless very decidedly marked are not dangerous. They should not exist in the water pats. If they do exist, however, to an appreciable extent, it denotes the presence of a too great proportion of expansives, which ordinarily is sufficient to condemn the sample.

Fig. 33 shows blotching, a pat which is usually indicative of either adulteration or under-burning. This condition in itself should not necessarily mean rejection, but should always induce an investigation of the causes producing it, which may or may not be sufficient to warrant rejection.

Fig. 34 shows pats which have left the glass (*A*) by mere lack of adhesion, (*B*) by contraction, and (*C*) by expansion. (*A*) is never dangerous

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*Presented to the authors by Mr. W. Purves Taylor.

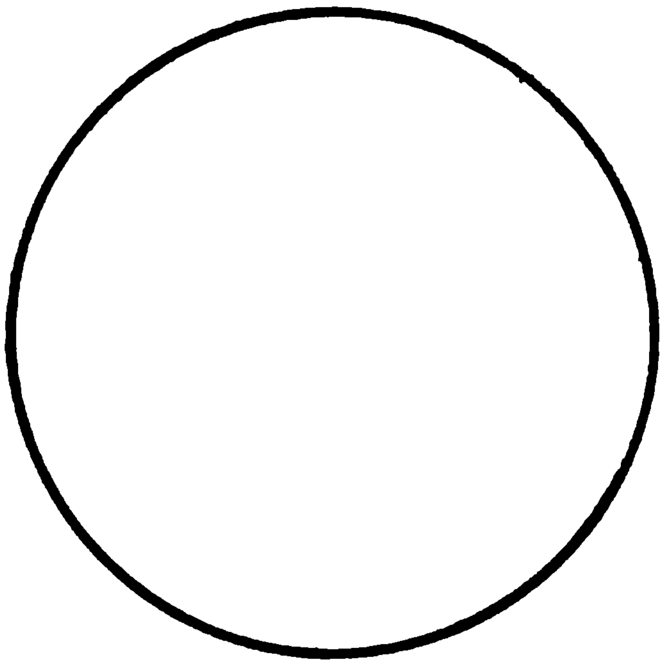

Fig. 30. Normal Pat.*

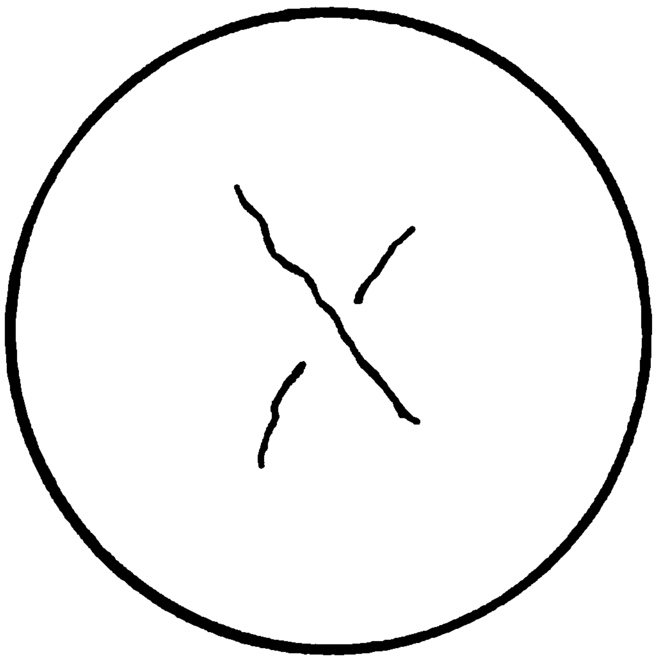

Fig. 31.—Harmless Shrinkage Cracks.*

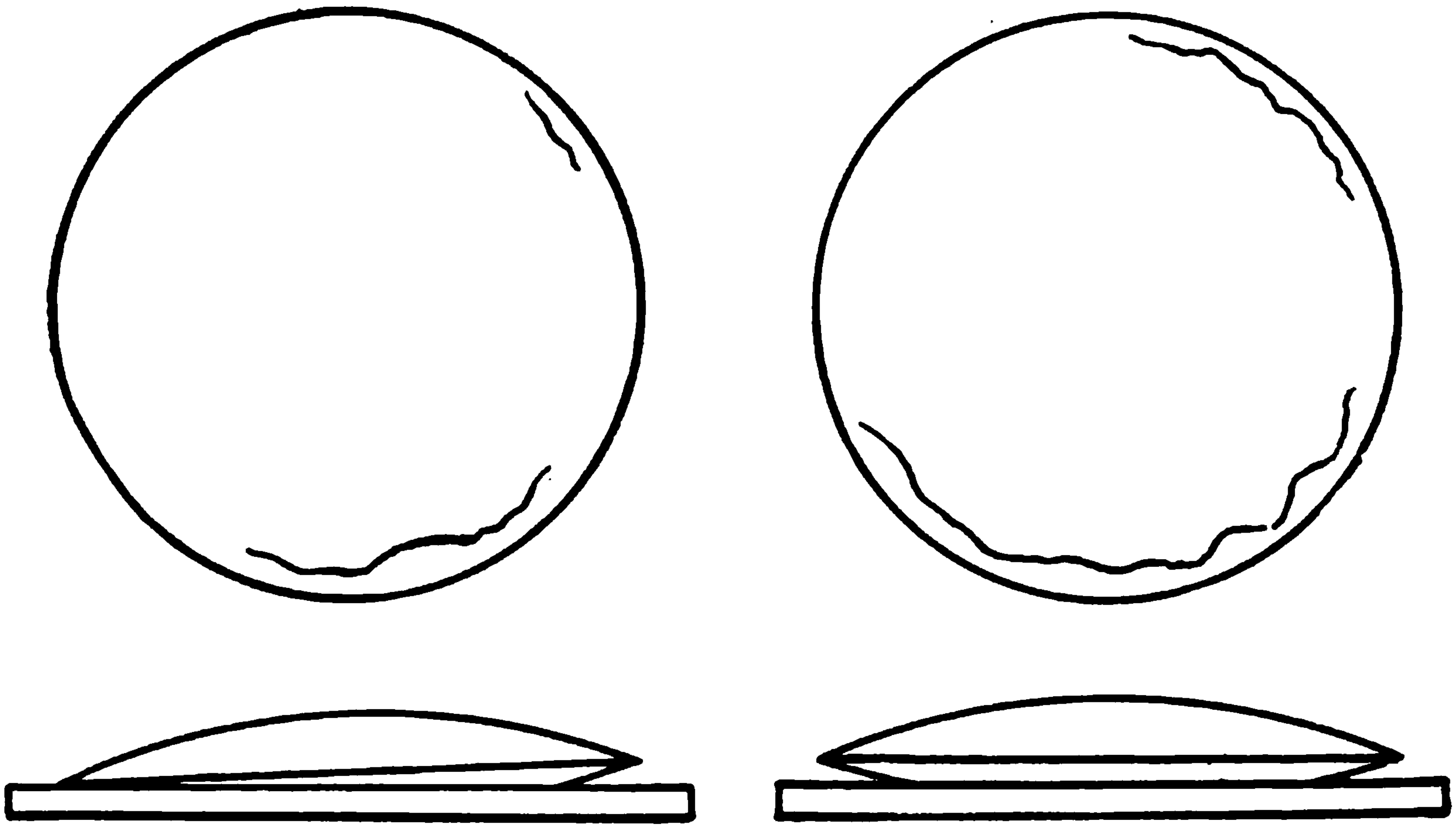

Fig. 32.—Expansion Cracks, Harmless in Air Pats.*

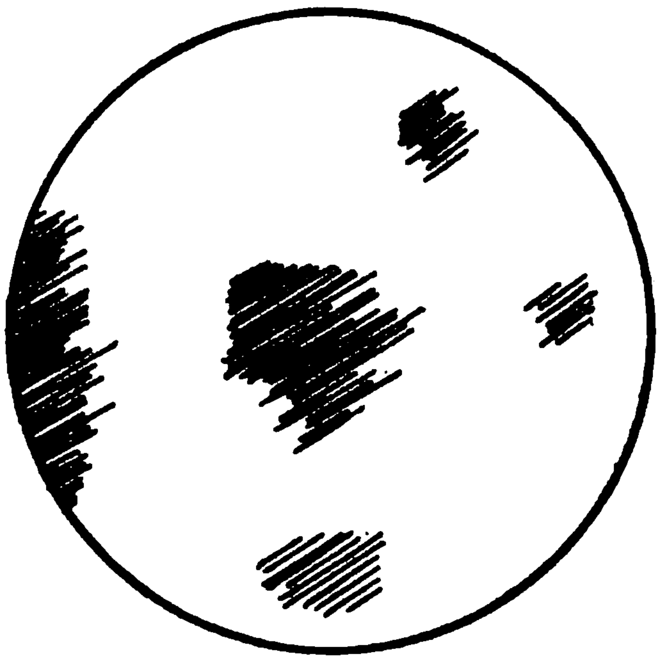

Fig. 33.—Blotches Requiring Investigation.*

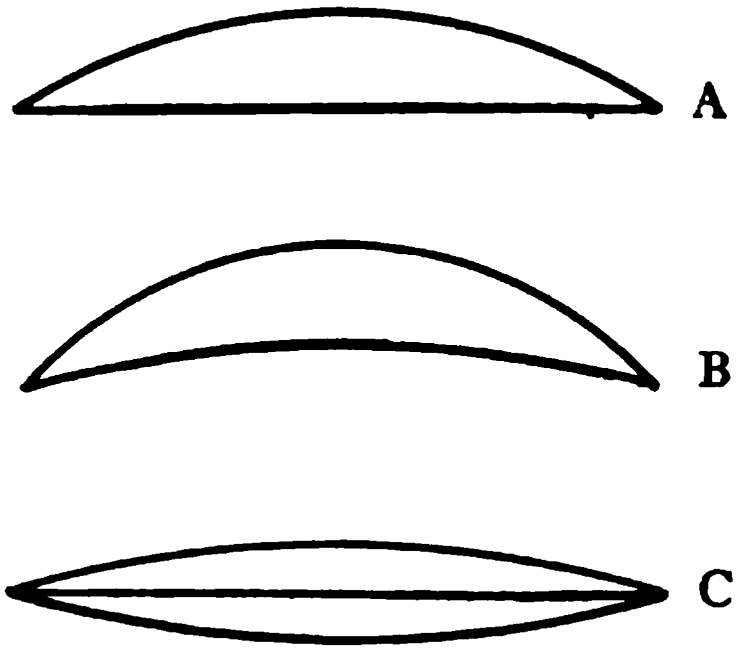

Fig. 34.—Pats which have left Glass.*

*See pp. 103 and 106.

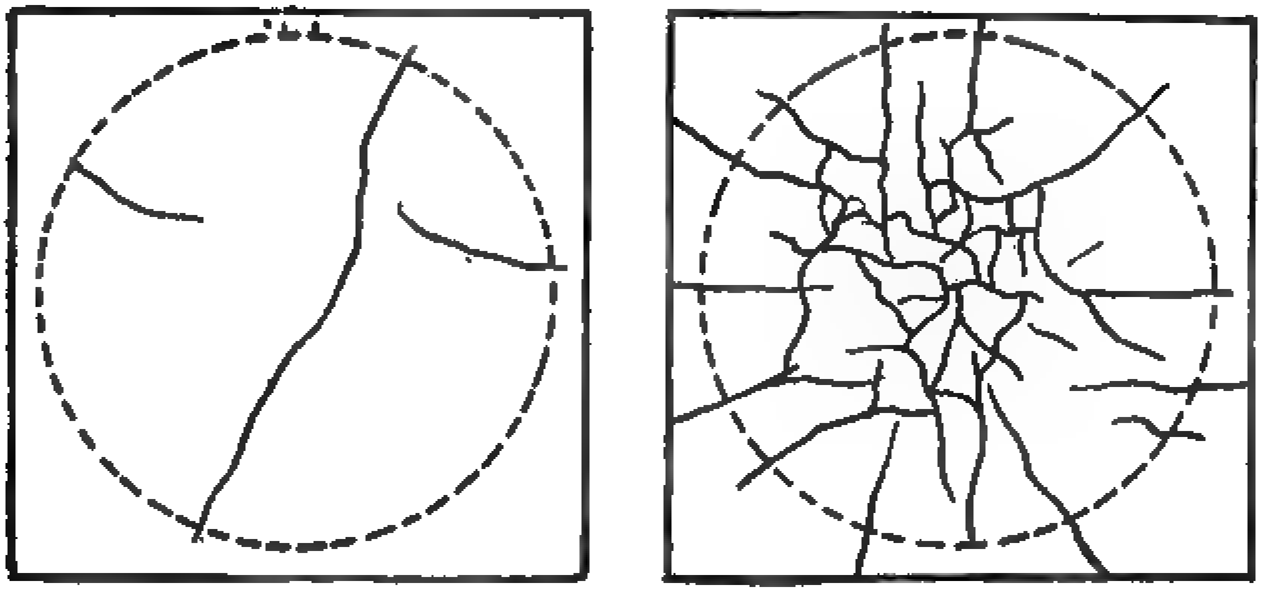

FIG. 35.—Cracked Glass (pat removed.)*

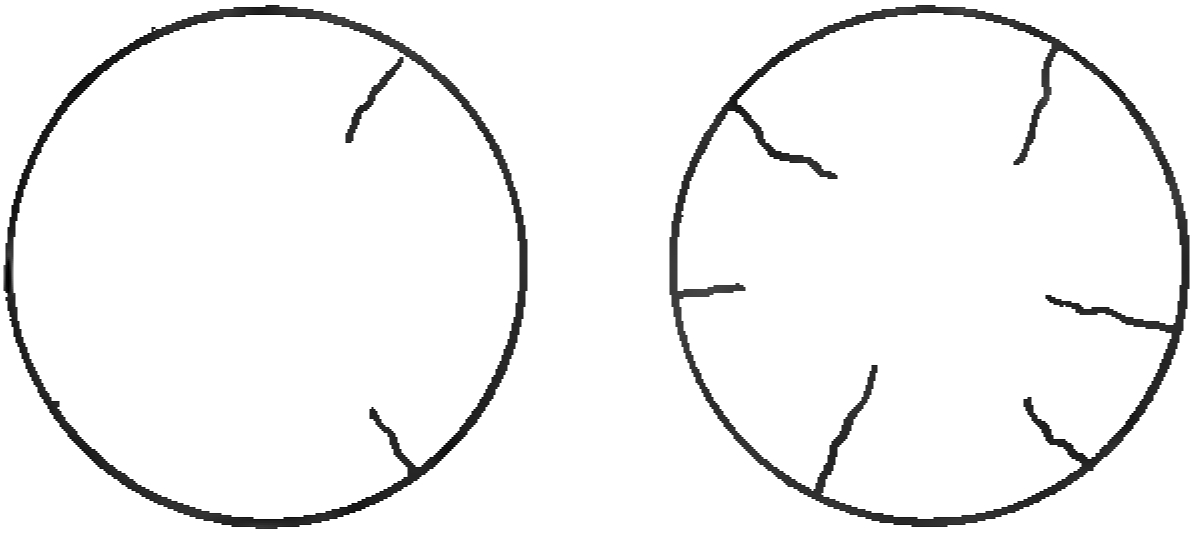

FIG. 36.—Incipient Disintegration.*

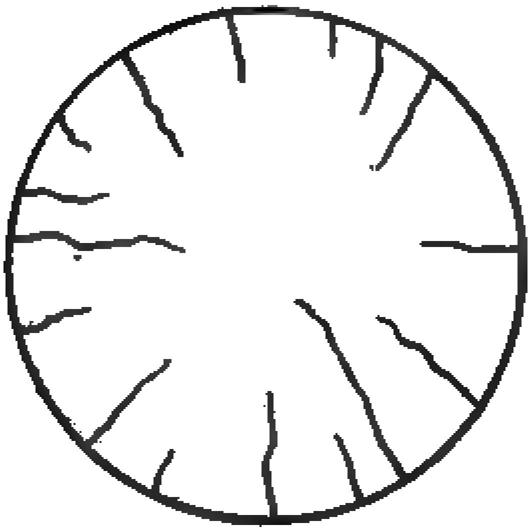

FIG. 37.—Complete Disintegration.*

*See pp. 103 and 106.

in either air or water. (*B*) and (*C*) are dangerous only when existing in a marked form. A curvature of about a quarter of an inch can be considered about the limit of safety in a 3-inch pat. Case (*C*) rarely, if ever, occurs in water.

Fig. 35 shows a peculiar condition in which the pat is perfectly sound and hard, but the glass on which it is made is badly cracked.* This has often been laid to chemical action, but this conclusion is doubtless erroneous. It is probably due entirely to expansion of the pat, when the adhesive strength of the cement to the glass exceeds the strength of the glass itself. It is only found in the water pats, and is not usually indicative of dangerous qualities of the cement.

Fig. 36 shows the radial cracks of incipient disintegration. These are the danger marks to be looked for in the normal pat tests, and are always sufficient to warrant rejection.

Fig. 37 shows cases of complete disintegration, which almost invariably begins merely by showing radial cracks, as in Fig. 36.

Accelerated or Hot Tests. The object of all forms of hot tests is to produce in a few hours the results which at a normal temperature require several days or perhaps months. Engineers are by no means agreed as to the value of accelerated tests, the chief objection to their use being that some cements which fail in these tests prove satisfactory in construction.

An argument for the hot test lies in the fact that Portland cement manufacturers are coming to recognize it as the very best test for them to use in determining whether their own cement will fulfil the requirements of permanent construction. In a recent letter to the authors the superintendent of one of the largest factories in the United States writes, "So far as we are concerned, we consider the hot test of the greatest importance. If this shows up well, we feel quite satisfied that all other tests will show up properly." Those desiring to investigate the various opinions upon the subject are referred to References, Chapter XXIX.

Mr. W. Purves Taylor, in a paper read before the Cement Section of the American Society for Testing Materials, at the Sixth Annual Meeting, 1903,† gives the results of a large number of accelerated tests made at the Philadelphia Testing Laboratory by boiling balls or pats (after 24 hours in moist air) for three or four hours, and the results of some of the conclusions there given are quoted as follows:

"The condition in a cement most affecting the result of an accelerated test is its age or the amount of seasoning it has undergone. Every cement,

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*Similar causes may produce one or two cracks in the glass.

†Proceedings American Society for Testing Materials, 1903, Vol. III, p. 374, also printed in *Engineering News*, July 23, 1903, p. 81.

no matter how well proportioned and burned, will contain at least a small amount of free or loosely combined lime, which will usually cause unsoundness if used or tested at once. This lime, however, will hydrate in a very short time on exposure to air, thus rendering it inert and preventing any expansive action. It will, therefore, be found in a large majority of cases that if a cement failing in the accelerated tests be stored for two or three weeks, this unsoundness will disappear, and the cement pass the test with ease."

This is illustrated in the following table and in Fig. 38, page 108, the first three photographs also showing various conditions which may be expected in specimens which fail to pass accelerated tests.

Effect of Age of Cement on Results of Boiling Test.

BY W. PURVES TAYLOR. (*See p.* 107.)

Age of cement when tested	TENSILE STRENGTH					NORMAL PAT TESTS		BOILING TEST
	Neat			1:3 sand				
	1 day	7 days	28 days	7 days	28 days	28 days in air	28 days in water	
1 week	550	765	762	171	225	Curled and soft.	Slightly checked.	Partly disintegrated.
2 weeks	548	67	771	170	246	Slightly curled.	Slightly curled.	Checked and cracked.
3 "	492	718	763	182	244	"O. K."	"O. K."	Slightly checked.
5 "	427	692	747	183	249	"O. K."	"O. K."	Sound.

"Coarseness of grinding is also a frequent cause of unsoundness for the reason that the larger particles are not readily susceptible to hydration, and contain for a long period of time expansive elements which very rapidly develop a disintegrating action when treated in the accelerated tests."

"A large number of tests on different cements were made and the time at which failure occurred was observed. In these tests it was found that of those samples which did not pass the test, 22% failed in the first half hour, 57% failed in the first hour, 85% failed in two hours, 96% in three hours, and 99% in four hours," "thus showing generally that a test piece of cement standing three or four hours of boiling will almost invariably stand a much greater length of time, and also that at least three or four hours should always be allowed for the test."

"Pats of cement allowed more than about twelve hours to harden will, if unsound, fail when tested by boiling at almost any time in the future."

"We now come to the very important question of the relation of the boiling tests to the other tests for soundness and strength as made in the

laboratory. No one who has had much experience with the boiling test questions that, although it is by no means infallible, the results obtained from it are generally corroborated by either the tensile tests or the normal tests for soundness. The writer has recently compiled some data in regard to this point, covering over a thousand tests on many varieties of cement, with the following results:

"Of all samples failing to pass the boiling test, 34% of them developed checking or curvature in the normal pats — or a loss of strength in less than twenty-eight days. Of those samples that failed in the boiling test but remained sound at twenty-eight days, 3% of the normal pats showed checking or abnormal curvature in two months, 7% in three months, 10% in four months, 26% in six months, and 48% in one year; and of these same samples, 37% showed a falling off in tensile strength in two months, 39% in three months, 52% in four months, 63% in six months, and 71% in one year. Or, taking all these together, of all the samples that failed in the boiling test, 86% of them gave evidence in less than a year's time of possessing some injurious quality.

One Week Old. Two Weeks Old.

Three Weeks Old. Five Weeks Old.

FIG. 38.—Specimens showing the Effect of the Age of the Cement upon its Soundness. (*See p.* 107.)

"On the other hand, of those cements passing the boiling test, but one-half of 1% gave signs of failure in the normal pat tests, and but 13% showed a falling off in strength in a year's time.

"This certainly makes a very strong showing in favor of the boiling test, at least considered from a laboratory standpoint.

"In order to show the great value sometimes obtained from the results of the boiling test, several examples are given in the table on page 110 of tests of cements occurring in the regular routine work of the laboratory."

The air and water pats of sample 2 of this table are shown in Fig. 39 at the age of four months. These pats were sound at twenty-eight days.

In conclusion Mr. Taylor lays special emphasis upon the fact that many cements which do not pass the boiling test will give excellent results in practice. He gives as the probable reason for this that the test for soundness is generally made immediately upon the receipt of a shipment, while the cement used in construction has opportunity to season, and upon the fact "that the disintegrating action of a cement is always far greater when mixed neat than when mixed with an aggregate, and the greater the amount of the aggregate the less the tendency to unsoundness." It is often good policy before rejecting a cement which fails to pass the hot test to hold it for a week or two so that it may further season and then retest it.

Fig. 39.—Examples of Unsound Pats at 4 months which were sound at 28 days. (*See p.* 109.)

Methods of Making Accelerated Tests. The methods of conducting accelerated tests are numerous, the object of all of them being to hasten

Evidences of Failure in Cement Indicated by the Boiling Test.

By W. Purves Taylor. (*See p.* 109.)

TENSILE STRENGTH							NORMAL PAT TESTS				
Neat				1:3 sand			Air		Water		Boiling test
1 day	7 days	28 days	4 months	7 days	28 days	4 months	28 days	4 months	28 days	4 months	
522	793	797	Disintegrated.	204	257	52	Very slightly curled; left glass.	Badly curled; soft and crumbly.	Left glass.	Disintegrated.	Disintegrated.
503	872	586	Disintegrated.	184	239	47	Very slightly curled; left glass.	Badly curled; soft and crumbly.	Left glass.	Disintegrated.	Disintegrated.
408	762	700	Disintegrated.	176	231	119	Very slightly curled; left glass.	Badly curled; soft and crumbly.	"O. K."	Disintegrated.	Disintegrated.
427	751	603	223	183	227	94	Very slightly curled; left glass.	Disintegrated.	"O. K."	Disintegrated.	Disintegrated.
503	827	717	177	220	252	132	Left glass.	Badly curled; soft and crumbly.	"O. K."	Disintegrated.	Disintegrated.
492	883	620	202	195	217	147	Left glass.	Badly curled; soft and crumbly.	"O. K."	Disintegrated.	Badly checked.
535	864	743	94	197	241	77	Very slightly curled; left glass.	Badly curled; soft and crumbly.	"O. K."	Badly curled and checked.	Checked and cracked.
502	829	722	320	203	247	65	Very slightly curled; left glass.	Badly curled; soft and crumbly.	"O. K."	Badly curled and checked	Checked and cracked.
	Neat tests not made.			172	219	93	Very slightly curled; left glass.	Badly curled; soft and crumbly.	"O. K."	Badly curled and checked.	Checked.
	Neat tests not made.			198	231	101	Left glass.	Badly curled; soft and crumbly.	"O. K."	Badly curled and checked.	Checked.

Note.—All of these cements were normal in specific gravity, time of setting, and fineness.

the hardening of the cement so as to produce in a few hours results which under ordinary conditions require weeks or months. Boiling the specimens, instead of steaming them as recommended by the Committee of the American Society of Civil Engineers, while more common, is more severe. Other methods are employed in Europe.

The Steam Test, recommended by the Committee of the American Society of Civil Engineers, requires, as already described (p. 77), that the pat after twenty-four hours in moist air shall be placed in an atmosphere of steam above boiling water.

The Boiling Test was originated by Prof. Tetmajer in Germany. After twenty-four hours in moist air, or until it is thoroughly set, the specimen is placed in cold water, which is raised to and then maintained at the boiling point for several hours. Three or four hours is the time specified by Mr. W. Purves Taylor, and often used in the United States, although some cement factories boil for twenty-four hours. Dr. Michaelis advocates six hours' boiling, and this period is specified by the French Commission.

Combined Boiling and Tensile Test. A regular test at many Portland cement factories consists in testing the tensile strength of briquettes which have been subjected to the hot test. A briquette of neat cement after twenty-four hours under a damp cloth is placed in an atmosphere of steam over boiling water for an hour or two, and then immersed in water at about the boiling point and boiled for about twenty-four hours, when it must show a certain tensile strength.

The Hot Water Test, as adopted by Mr. Henry Faija in England, and advocated there by Mr. David B. Butler, consists in subjecting a newly mixed pat to a moist heat of 100° to 105° Fahr. (38° to 40° Cent.) for six or seven hours, or until thoroughly set, and then placing it in warm water at a temperature of 115° to 120° Fahr. (46° to 49° Cent.) for the remainder of the twenty-four hours. Mr. Deval in France employed a temperature of 176° Fahr. (80° Cent.) for a period of six days.

Other Accelerated Tests which have been employed in Europe are oven tests, where the specimen is heated in an oven; glow tests, where a ball is heated over a gas flame, and Prussing disc tests, where discs are formed under heavy pressure and then exposed to hot water.

Measurement of Expansion. Appliances have been devised for testing the soundness of cement by measuring the amount of expansion or deformation which it undergoes in different periods of time. The principal of these are the long bar apparatus, devised by Messrs. Durand-Claye and

Debray, which was recommended by the French Commission, Bauschinger's caliper apparatus, and Le Chatelier's tongs.*

The Chimney Expansion Test, in which a small quantity of neat cement is solidly pressed into a straight lamp chimney with the idea that an unsound cement will break the glass, is worthless, as all first-class cements expand to a greater or less degree.

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*Described in Spalding's Hydraulic Cement, 1903, p. 166.

CHAPTER VIII

SPECIAL TESTS OF CEMENT AND MORTAR

The most important tests for comparing the qualities of different cements and for determining their practical value have been described in the preceding chapter. Certain other tests are often made to investigate special qualities of a cement or mortar, or for scientific research.

Such special tests may be enumerated as follows:

Color.
Weight.
Microscopical.
Compressive.
Transverse.
Adhesive.
Shearing.
Abrasive.
Porosity.
Permeability.
Yield of mortar.
Rise in temperature.

COLOR

The color of a cement bears but slight relation to its quality, but a variation of color in the same brand is sometimes an indication of inferiority. Natural cements made in different localities may often be distinguished from each other and from Portland cements by their color.

Portland Cement. The chemical composition of Portland cements made by different processes is so uniform that the color of different brands varies less than that of Natural cements.

The color of Portland cement is described as a cold blue gray. In England the term "foxy" is applied to a Portland cement of a brownish color. According to Mr. David B. Butler* this denotes "insufficient calcination or the use of unsuitable clay or possibly excess of clay." He further states that if a Portland cement contains a large quantity of underburned particles, on account of their lower specific gravity they tend to rise to the surface on troweling, thus forming a yellowish brown film which is noticeable in the section of the briquette after fracture.

*Butler's Portland Cement, 1899, p. 255.

The dark color of the coarser particles of a Portland cement left as residue on a screen is due simply to the fact that cement clinker is black, and pieces which are not finely ground retain the color of the clinker.

Natural Cement. The color of Natural cement varies with the character of the rock and consequently with the locality in which it is produced. It ranges from the light écru of the Utica (Ill.) cement to the dark grayish brown of the Rosendale (N. Y.). Samples received by the authors from various manufactories show the James River cement to be a light yellowish brown, the Akron (N. Y.) cement, écru, the Milwaukee (Wis.) cement, drab, and the Louisville (Ky.) cement, a brownish gray. Certain other brands are similar in color to Portland.

Puzzolan Cement. Puzzolan cement made from slag is of a light lilac shade, much lighter than Portland. After being kept under water it assumes, when freshly fractured, a bluish green tint. This green tint, which according to Candlot* is due to sulphide of calcium present in the cement, is especially noticeable in a sample kept in sea water, and fades on exposure to dry air.

WEIGHT OF CEMENT

Weight is no indication of quality. Formerly, nearly all specifications required that a cement should reach a certain standard of weight per struck bushel or per cubic foot, on the principle that, other things being equal, a thoroughly burned cement is heavier than one which is underburned. But when, on the other hand, the degree of fineness was found to affect the weight much more than any difference in calcination, the worthlessness of this requirement became apparent, and the test for specific gravity was substituted.

The following table by Eliot C. Clarke† illustrates the difference in weight between cements of the same manufacture which contain different percentages of coarse particles.

Weights of Cements Containing Varying Percentages of Coarse Particles. (See p. 114.)

BY ELIOT C. CLARKE.

Percentage of cement retained on No. 120 sieve	Weight per cu. ft.
0	75 lb.
10	79 "
20	82 "
30	86 "
40	90 "

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*Candlot's Ciments et Chaux Hydrauliques, 1898, p. 159.

†Transactions American Society of Civil Engineers, Vol. XIV, p. 144.

Mr. Henry Faija's experiments* arranged in the following table prove that the weight of a cement decreases with age. His explanation for this is that the lower specific gravity of the moisture and carbonic acid absorbed from the air tends to increase the bulk of the cement without correspondingly increasing its weight.

Decrease of Weight of Cement with Age. (*See p.* 115.)

BY H. FAIJA.

	Weight per cu. ft. lb.	Percentage of loss in weight per cent.
When received	88	..
After one month	85½	2.7
" three months	79½	9.9
" six "	78	11.4
" nine "	75½	14.2
" one year	74	15.9

Method of Weighing Cement. The apparatus finally recommended by the French Commission, after a series of tests by Mr. P. Alexandre,† was a circular funnel with screen, as shown in Fig. 40. The cement placed upon the screen is stirred with a wooden spatula 4 cm. ($1\frac{5}{8}$ in.) wide, and 25 cm. (10 in.) long, and falls through the screen into the cylindrical measure of one liter capacity (0.61 cu. ft.).

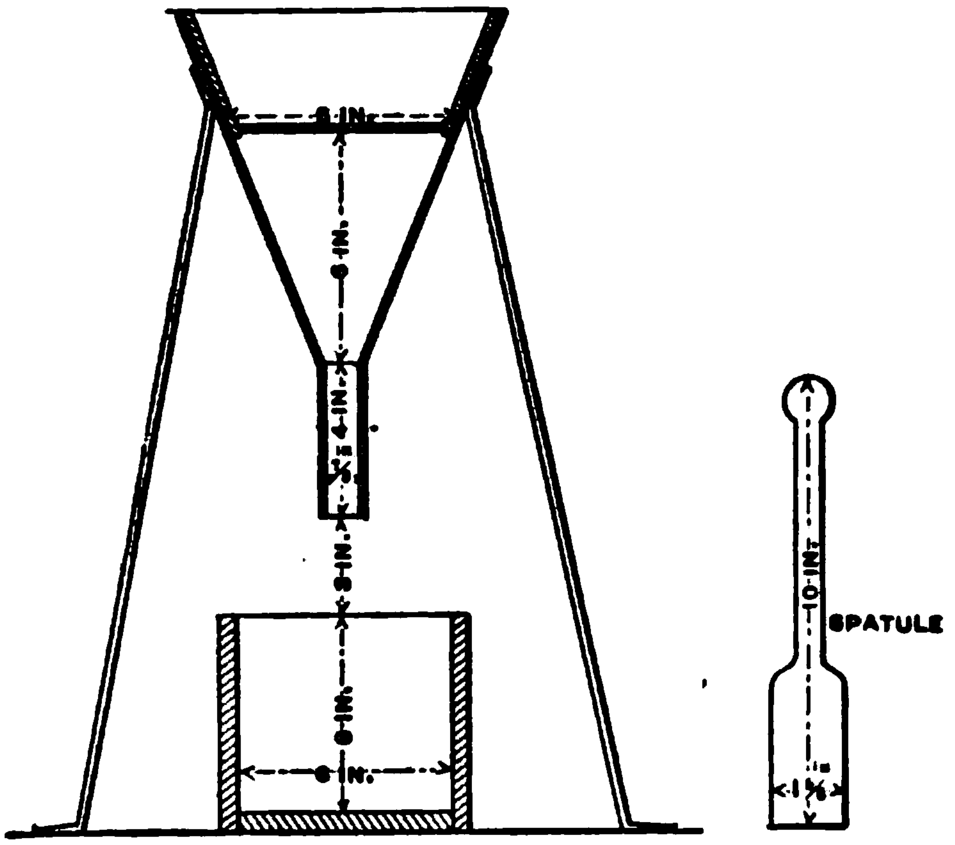

FIG. 40. Funnel Used in Weighing Cement. (*See p.* 115.)

MICROSCOPICAL EXAMINATION OF PORTLAND CEMENT CLINKER

The structure of Portland Cement clinker can be clearly discerned with the aid of the microscope and polarized light by preparing thin sections of it in the same way as those of rocks made by petrographers.

Le Chatelier, a French engineer, and Törnebohn,

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*Butler's Portland Cement, 1899, p. 240.

†Commission des Méthodes d'Essai des Matériaux de Construction, 1895, Vol. IV, p. 21.

a Swedish petrographer, some years ago identified two essential mineral entities, and three others of less importance, as constituents of Portland cement clinker. Törnebohn denominated the two essential constituents alite and celite.

Mr. Clifford Richardson has within the last few years taken the subject up very elaborately in this country, and his results go to show that optical methods of examining clinker will eventually prove of great interest, not only in determining the character of clinker, but also in pointing out means of improving the methods of production.

COMPRESSIVE TESTS OF CEMENT

Compression testing machines are coming into general use in America. For merely determining the quality of a cement, tensile tests are more convenient because they can be made more quickly and require less powerful machines, but for comparing different sand aggregates and for its adaptability to testing concrete by compression or by transverse, *i.e.*, beam, tests, the compression machine possesses great advantage. The French Commission recommend compression tests in addition to tension, and many engineers in the United States advise them in well equipped laboratories.*

Types of Compression Testing Machines. Machines especially adapted for compressive tests are built with capacities ranging from 30 000 to 400 000 lb., or even larger. The Emery Machine at the Watertown Arsenal, U. S. Army, is of 800 000 lb. capacity. A machine with a capacity of not less than 40 000 lb. is required for 2-inch cubes of neat cement or mortar, while for 6-inch cubes of mortar or concrete a machine should run to at least 150 000 lb. Nearly all compression machines may be arranged for tension or transverse tests by the adjustment of special appliances, although they are too cumbersome for testing ordinary cement briquettes.

A testing machine driven by power is illustrated in Fig. 41. This same type, up to 60 000 lb. capacity, is built for hand operation, and in larger sizes may be arranged to run by belting, when it is sometimes fitted with appliances for automatically recording the elastic limit and the breaking load.

An American machine of the same type as the German Amsler-Laffon compression testing machine is illustrated in Fig. 42, page 118. The hydraulic

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*Proceedings American Society of Civil Engineers, April, 1900, p. 125.

power is applied by turning the hand wheel and the load is read directly from the pressure gage.

Form of Compression Specimens. Extended tests were made for the French Commission by Mr. P. Siméon[*], in which he employed specimens of various shapes and sizes, and compared the results with those obtained from crushing the halves of briquettes which had been broken in tension. Quoting from a discussion of Mr. Thompson[†] upon the Report of the Cement Committee of the American Society of Civil Engineers:

FIG. 41.—Compression Testing Machine. (*See p.* 116.)

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[*] Commission des Méthodes d'Essai des Matériaux de Construction, Vol. IV, 1895, p. 187.

[†] Sanford E. Thompson in Proceedings American Society of Civil Engineers, August, 1903, p. 646.

FIG. 42.—Hydraulic Compression Testing Machine. (*See p.* 116.)

The Commission reached the conclusion that the briquettes which had been broken in halves by tension should be used for the compressive tests. The two halves of each briquette are crushed separately and the sum of the two results divided by the total area of the briquette, thus obtaining the compressive strength per unit of surface. The surface area of the United States standard briquette recommended by our Committee is almost exactly 4 sq. in. Instead of the halves of a briquette, a single cylinder having the same thickness and the same area of surface as a whole briquette may be used with substantially equivalent results.

Specimens which are rough or uneven may be smoothed by gentle rubbing on a stationary grindstone.

In breaking, the pressure should increase uniformly, and at such speed that it will require between one and two minutes to crush each specimen.

For comparing the strength of cement paste or mortar, with that of other materials which cannot readily be molded in cement molds, the Commission recommends cubes having an area of 50 sq. cm. (7.75 sq. in.) on each face. For a United States standard, cubes 2 in. on an edge, that is, with all faces having an area of 4 sq. in., conform to most common usage, and are therefore best for this class of comparative tests.

A mold for cubes is shown in Fig. 43.

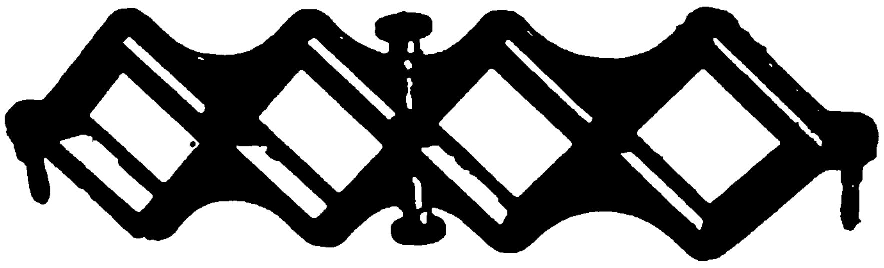

FIG. 43.—Gang Mold for Compression Cubes. (*See p.* 119.)

Relation of Compressive to Tensile Strength. Mr. R. Feret* concludes, after an extended series of tests, that there is no constant relation between resistances to compression and tension. He also concludes that the rate of increase in strength varies with the different cements, so that "two different mortars having the same resistance to compression do not necessarily break under the same tension." He claims that compression tests give better results than tension and furnish "the real measure" of the cohesion of mortars. These opinions are generally corroborated by cement experts.

The ratio of tension to compression also varies with the character of the sand or other aggregate. With a larger proportion of cement the compressive strength increases faster than the tensile strength, thus giving a higher ratio. This law continues to hold with concrete of different proportions, that containing the largest proportion of aggregate showing the highest compressive strength in comparison to its tensile strength.

*Bulletin de la Société d'Encouragement pour l'Industrie Nationale, 1897, Series 5, Vol. II.

A comprehensive series of compressive and tensile tests of mortars mixed in various proportions is given in the table on page 136.

Prof. J. B. Johnson* (from experiments of Prof. Tetmajer in Germany, extending from 7 days to one year) has deduced an approximate formula for the relation of compressive to tensile strength of Portland cement mortar at different ages. Plotting the ratios for different ages he finds the equation of the resulting curve to be

$$\frac{\text{Compressive strength}}{\text{Tensile strength}} = 8.64 + 1.8 \log. A,$$

where

$$A = \text{age of the cement mortar in months.}$$

By this formula it will be seen that the ratio varies from 8.6 on a one-month test up to 10.6 on a 12-months test.

TRANSVERSE TESTS OF CEMENT

Transverse, or flexion, tests of beams or prisms while very convenient for concrete are now seldom used for testing the quality of cement, although Gillmore and other of the older experimenters largely employed this form of test. Transverse tests are of value in comparing the relation between fiber stress and tension, and with proper care may give as uniform results as tension tests. As is stated below, the fiber stress bears a definite relation to the tensile strength, but since there is no fixed relation between tension and compression, there can be no fixed relation between transverse strength and compressive strength. Compression testing machines (see Figs. 41 and 42, pages 117 and 118) may be adapted for transverse tests by a suitable arrangement of supports and knife edges.

Size of Specimen. Mr. Durand-Claye† made for the French Commission an extended series of tests by flexion or bending. As a result of his report, the Commission adopted for this form of test square prisms 12 cm. (4.72 in.) long by 2 cm. (0.79 in.) on a side.

In breaking, a prism is placed on its side — that is, on a face which has been in contact with the mold — upon two knife-edges, 10 cm. (3.94 in.) apart, and the load is applied at the center through a slightly rounded knife-edge. The load should be applied continuously at the rate of 1 kgr. (2.2 lb.) per second. The same number of specimens should be broken as in tensile tests, and the results averaged.

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*Johnson's Materials of Construction, 1903, p. 419.

†Commission des Méthodes d'Essai des Matériaux de Construction, 1895, Vol. IV, p. 211.

English measure will naturally change the dimensions of the specimen to 1 by 1 by 6 in., to be broken upon knife-edges 5 in. apart.*

A prism 2 by 2 by 8 in. was employed by General Gillmore in experiments described in his famous "Treatise on Limes, Hydraulic Cements and Mortars," and has been adopted by other American engineers, but with apparatus of sufficient delicacy there is no reason why the specimens need be larger in section than tensile specimens, and the dimensions of 1 by 1 by 6 inches suggested above are recommended for comparative tests of neat cements and mortars. A form of mold is shown in Fig. 44.

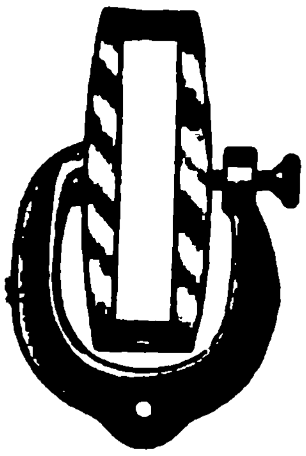

FIG. 44. — Mold for Prism. (*See p.* 121.)

Relation of Tensile to Fiber Stress. In the experiments mentioned above Mr. Durand-Claye compared all of his tests for flexion with tensile tests of briquettes made and tested at the same time. As a result, he obtained as the ratio between the ultimate fiber stress in flexion and the tensile strength, 1.92 at 7 days and 1.86 at 28 days; or in round numbers, 1.9 for both. That is, tensile fiber stress is 1.9 times the simple tensile stress of the same material. In this connection he calls attention to the fact that a briquette tested in tension gives a result less than the real resistance, while a prism tested in flexion gives a greater result. He judges that the real resistance may be approximated by taking the mean of the two results.

Mr. Durand-Claye also found the mean error by the two methods of testing to be very similar, with tensile briquettes the variation being about 2.02 % as compared with 2.52% variation in the flexion tests. In tests with mortar there was less variation with prisms than with briquettes.

Prof. Edgar B. Kay states that in recent experiments he has obtained more uniform results with tranverse than with tensile tests.

Comparative tests of Mr. R. Feret in tension, flexion, and compression are shown in the table on page 136.

ADHESION TESTS OF CEMENT

Mr. E. Candlot† made a large number of tests of adhesion for the French Commission, and designed a mold adopted as the French Standard. With reference to such tests he says that since the adhesion of mortar to a stone depends upon the state of the surface and the nature of the cement,

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*Sanford E. Thompson in Proceedings American Society Civil Engineers, August, 1903, p. 646.

†Commission des Méthodes d'Essai des Matériaux de Construction, 1895, Vol. IV, p. 281.

absolute tests are of little value, but comparative tests, if made under identical conditions, are of real interest to the builder.

Thus, to cite several examples, the tests of adhesion prove that a mortar regaged after having set possesses a strength of adhesion much smaller than the same mortar gaged and put in place before its set, the resistance to tension and compression of these two mortars remaining, however, almost the same; that mortars gaged dry have a more feeble adhesion than mortars gaged slightly liquid; that mortars gaged with an excess of water have in tension a resistance less than their adhesive strength, etc.

Method of Making Adhesion Tests. In the same report Mr. Candlot describes the forms of specimens suggested by Dr. Michaelis and others, and then presents a form which he considers to best meet the requirements. On account of the difference in section of the French standard briquette, the mold he describes is not suitable for making specimens to fit the clips

Fig. 45.—Mold for Adhesion Block. (*See p. 122.*)

on American testing machines. To adapt his mold to American standards, the authors have designed the mold shown in Fig. 45. The method of making tests is described by Mr. Thompson* as follows:

Adhesion is considered by Mr. Candlot in two ways: First, with reference to the relative adhesive qualities of different cements; and, second, with reference to the adhesion of the same cement mortar to other materials of different natures. The same general method is advocated in both cases.

Briquettes are formed, as described below, of a shape which can be broken in an ordinary tensile testing machine. The European tensile briquette is of small section, 5 sq. cm. (0.775 sq. in.), and of an inconvenient shape for molding in halves. The area of the breaking section is therefore doubled by the Commission, while the curves where the clips take hold remain the same, so that the distance between the two points of each clip is unchanged. The shape of the United States standard briquette is such that fewer changes have to be made in its outline, and the regular section of 1 sq. in. need not be altered.

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*Proceedings American Society of Civil Engineers, August, 1903, p. 647.

The Commission found that adhesion briquettes could not be molded satisfactorily in the manner used for tension briquettes. They advised finally a mold in which a half briquette could be made, and then when this had set, the same mold could be used for completing the other half. In Fig. 45 is shown the style of mold selected, but with the dimensions changed to adapt the briquette to the United States standard form of clip. It consists of a bottomless box, which divides vertically in the center on the line *BB*, so that the half briquette can be removed readily. The bottom is formed of a movable bronze plate, shown at *A*.

For the first class of tests, to determine the relative adhesion of different cements, a normal adhesion block is formed of a mortar composed, by weight, of 1 part of highest quality Portland cement, which has passed a No. 75 sieve, and 2 parts of fine sand, gaged 9% of water. As soon as it is rammed into the mold, the mold is removed, and after remaining in moist air for 24 hours the half briquette is placed in water until it is required. It must set for at least 28 days. When required for use, the block is dried and the surface polished with emery paper. The block is then placed on a table with the large end down, the half mold, with the disc *A* removed, set on top of it and filled with plastic mortar consisting of the cement which it is desired to test mixed with sand in the required proportions, thus completing the briquette. This briquette is treated and tested as an ordinary tension specimen.

For the second class of tests, if the material can be molded, it is formed as a half briquette, and the specimen completed with the mortar to be tested. If solid, a plate of the material, several millimeters thick, having one smooth face, is prepared, and placed at the bottom of the mold, on top of the bronze plate, and the first half of the specimen is formed by filling the mold with neat cement. After setting, the half of the briquette is completed with the mortar which it is desired to test.

Adhesive Strength of Mortar. The following table from tests of Mr. Candlot, presented to the French Commission,* shows the results of adhesive tests upon Portland cement mortars cemented to the normal adhesion block by the method described in the preceding paragraphs. It is noticeable that, in the same column, the values, each of which represents a single specimen, are fairly regular, but that there is a very great variation in the adhesive strength of mortars made from different cements, and no uniform relation between the strength of mortars of different proportions.

Adhesion of Mortar to Various Materials. The results of tests made by Professor Tetmajer in Germany, quoted by Mr. E. Candlot, are briefly as follows: 1:2 Portland cement mortars cemented to sandstone gave an adhesive strength after 28 days of from 5.5 to 8.8 kg. per sq. cm. (78 to 125 lb. per sq. in.). To rough glass the adhesion was about 3.5 kg. per sq. cm.

*Commission des Méthodes d'Essai des Matériaux de Construction, 1895, Vol. IV, p. 285.

(50 lb. per sq. in.). Tests made at Boulogne-sur-Mer using blocks of marble showed, after 28 days, variations of 3.1 to 8.3 kg. per sq. cm. (44 to 118 lb. per sq. in.). Regaged mortar showed about half the strength in adhesion of fresh mortar.

*Adhesive Strength of Portland Cement Mortars in Pounds per Square Inch.**

BY E. CANDLOT.

Cement.	A				B		C		D	
Proportions of mortar.	1:3	1:3	1:2	1:2	1:3	1:2	1:3	1:2	1:3	1:2
Per cent. of water.	12	13.8	9.5	15	12	13	15	17	12	13
	lb	lb.	lb.	lb.	lb.	lb	lb.	lb.	lb.	lb.
7 day tests.	107	135	142	149	36	36	36	43	60	65
	195	131	145	152	36	43	38	50	60	65
	131	156	135	128	28	36	38	36	57	71
	156	152	135		28	50		36	67	82
					36				57	107
Average. ...	147	143	139	143	33	41	37	41	60	78
28 day tests.	164	192	188	178	92	142	78	95	152	95
	178	206	294	152	81	114	74	78	128	88
	178	220	124	192	85	114	71	117	114	74
	199	220	185	156	60	85	81	100	107	
					67	102		88		
Average. ...	180	209	198	169	77	111	76	96	125	86

Mr. E. S. Wheeler† has made several series of tests, inserting thin discs of different materials in the center of briquettes. Although the irregularity in the results cast considerable doubt upon his method of testing, the experiments tended to show that the adhesive strength to sawn limestone of Portland cement mortar in proportions 1:0 to 1:2 is about one-third the cohesive strength of the mortar alone. Mr. Wheeler concluded that grooving the surface of the stone has no appreciable effect on the adhesive strength. For the maximum adhesive strength more water is required than for the maximum cohesive strength even if the surface of the stone be saturated. The substitution of a small portion of lime for a part of the cement apparently increases the adhesive strength.

*Molded upon normal adhesion blocks, see pp. 122 and 123.

†Report Chief of Engineers, U. S. A., 1895, p. 3019 and 1896, pp. 2799 and 2834.

Mr. R. Feret* states that adhesion to stone increases as the stone becomes more porous. He found, as did Mr. Wheeler, that irregularities of surface of the stone do not seem to affect the adhesive strength. With iron, however, roughening the surface increases the adhesion of the mortar. A dirty surface or insufficient moistening of the surface lowers the adhesion.

The method adopted by various experimenters of crossing two bricks and cementing them together, then determining the loads required to separate them, is obviously inaccurate because of the difficulty of distributing the pull uniformly over the entire surface.

The adhesion of mortar to iron or steel is of such practical importance in the use of iron or steel for reinforcement, and the setting of bolts in mortar and concrete, that the subject is discussed in connection with reinforced concrete in Chapter XIV.

SHEARING TESTS OF CEMENT AND MORTAR

Mr. R. Feret made a series of shearing tests upon different mortars which are quoted in column (20) of the table on page 136. He employed for the shearing test the halves of small prisms which had been broken to determine the transverse strength, placing the specimens and loading them as is shown in Fig. 46.

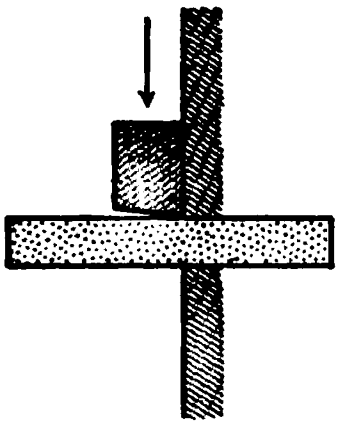

FIG. 46.—Shearing Test. (*See p.* 125.)

ABRASION

Abrasion or wearing tests have been made by pressing the specimen against a grindstone, an emery wheel, or a cast-iron disc, the last requiring sand in definite proportions to be poured upon it to increase the friction.

Tests by Mr. Eliot C. Clarke† tend to indicate that for Portland cement mortar the best proportions to resist abrasive forces are 1 : 2 and for Natural cement mortar 1 : 1, the resistance of Portland cement mortar mixed with two parts of sand being nearly double that of both the richer 1 : 1 mixture and the leaner 1 : 2½ mixture.

POROSITY TESTS

The determination of the porosity of a specimen is often necessary in scientific research and for comparing the relative absorptive properties of

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*Communication au Congrès de Budapest, 1901.

†Transactions American Society of Civil Engineers, Vol. XIV, p. 167.

building materials. Porosity is a passive quality referring to the actual voids, *i.e.*, air and uncombined water in a substance as distinguished from permeability or percolation, the quality of a substance which permits the flow of a liquid or gas through it.

Method of Testing Porosity. Messrs. P. Alexandre, P. Debray, and H. Le Chatelier* recommended a method for making the test for porosity which, with the units converted into English measure, is summarized by Mr. Thompson† in his "Discussion on the Report of the Committee of the American Society of Civil Engineers on Uniform Tests of Cement." This method is suitable for testing the porosity of concrete as well as of mortar.

The porosity of a mortar is expressed as the ratio or percentage of voids to the total volume. In measuring the voids all water in the mortar is included except that of crystallization.

If

V = total apparent volume of mortar;
v = volume of solid portion of mortar;
v' = volume of voids in mortar;

then

$$\text{Porosity} = \frac{v'}{V} = \frac{V-v}{V}.$$

The size of specimen recommended is that having a volume of between 0.3 and 0.5 liter (18 to 30 cu. in.).

The solid volume, v, is found by the application of the principle of Archimedes, that the difference between the weight of a body in air and its weight when suspended in a liquid is equal to the weight of the liquid displaced. From the weight of the displaced liquid, its volume, which is manifestly the volume, v, of the mortar, can be readily calculated.

In English measure, if

P = weight of specimen after drying;
p = weight suspended in water after saturation;
W = weight of 1 cu. ft. of water;
v = volume of solid portion of mortar;

then

$$v \text{ (in cubic feet)} = \frac{P-p}{W}.$$

In order that the specimen may be thoroughly impregnated with water and all air driven from the pores when determining p, its weight in water, the specimen is first exposed for ½ hour in rarefied air at a pressure not greater than 25 mm. of mercury. Water is made to cover it, and then the atmospheric pressure is restored. It must now remain in the water 24 hours before being weighed. If apparatus for rarefying the air is not at

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*Commission des Méthodes d'Essai des Matériaux de Construction, 1895, Vol. IV, p. 247.

†Proceedings American Society of Civil Engineers, Aug. 1903, p. 648.

hand, and if the specimen will stand exposure to heat, an alternate method may be used. The specimen, after 48 hours in water, is placed in cold water, raised to boiling, and boiled for 2 hours, then allowed to cool for 24 hours. The weight, *P*, used in this determination, is taken after exposing it to a heat of between 40° and 60° Cent. (104° and 140° Fahr.), until there is no loss in weight, care being taken to prevent any access of carbonic acid gas from the heating apparatus.

The apparent volume, *V*, of the specimen, can be found by direct measurement, or by calculation from its loss of weight in water, using again the principle of Archimedes. To prevent saturation in the later proceeding, it can be covered with a thin coating of grease spread with the fingers. The weight in water must be taken before that in air.

The standard test of porosity is made with 1: 3 mortars of normal plastic consistency, 28 days old. Other proportions and ages suggested are 1: 2, and 1: 5, at 7 days, 28 days, 6 months and 1 year.

The Porosity of Different Mortars. Porosity includes the voids or pores occupied by both air and water, the relative volumes of the two classes of voids varying with the freshness of the mortar.

In different fresh mortars there is much less variation in the volume of air voids than is generally supposed. If we leave out of consideration mortars that are mixed to such a dry consistency that voids are apparent to the eye, we notice from column 10 of the table on page 136 that in mortars proportioned richer than 1: 5 the air voids rarely exceed 12%, and for most mixtures are in the neighborhood of 4% to 8%, that is, 4% to 8% by volume of air is entrained when gaging. Although experiments of Messrs. Candlot* and Alexandre show similar results, the authors, by mixing the materials with gloves, as recommended by the American Society of Civil Engineers, and using more water than required for standard consistency, — about, in fact, the consistency used by stone masons, — have obtained mortars in proportions of cement to either standard sand or bank sand of 1: 0, 1: 1 and 1: 2 with no appreciable entrained air, and leaner mixtures with 1% to 2% air. A few experiments carefully made tend to show that in larger batches thoroughly mixed to soft consistency these low percentages may be obtained. Such mortars require no ramming, in fact the volume cannot be reduced after it is carefully introduced into the measure, except that if a very wet mixture is used it will expel a portion of its surplus water when setting so that the volume set is less than the volume green. One would naturally expect a greater variation with different materials and different proportions of water, but as a matter of fact, in a fresh mor-

*Candlot gives a range of from 2 or 3% for mortars of coarse sand, up to 10% with fine sand.

tar slightly softer than standard consistency, the spaces between the particles of sand and cement are not occupied by air but by water.

As the mortar dries after setting, the variation between different mortars is more appreciable, since the additional amount of water which is required with mortars of fine sand partially evaporates and leaves air voids. It is evident from experiments by Mr. Alexandre that the percentage of air voids due to evaporation of water ranges from 7% with a very coarse sand to 18% with a very fine sand. Assuming a small allowance for entrained air in the fresh mortar, due to imperfect mixing, we may estimate a range of from 7% to 25% total air voids in mortar after setting and drying. An average mortar of Portland cement and fairly coarse bank sand, in proportions 1 : 2 by weight or 1 : 2½ by volume, from experiments of the authors, may be expected to contain about 10% of air voids after setting and hardening, and to have a total porosity of about 25%. The porosity of well proportioned concrete is much lower (see p. 417). The porosity is but slightly affected by the percentage of water used in gaging, because an excess of water rises to the surface. (See p. 416.)

PERMEABILITY OR PERCOLATION TESTS

The permeability of mortar and concrete is discussed and the laws which govern it formulated in Chapter XX, page 416. Permeability is distinguished from porosity on page 126.

Method of Testing Permeability. When preparing its final report, the French Commission* first experimented with cylindrical blocks having in the center a truncated well into which a vertical tube was introduced for a short distance to convey the water under pressure. They finally recommended instead of this form a cube of cement or mortar with a pipe cemented to its upper surface. Quoting again from Mr. Thompson's Discussion† on Uniform Tests of Cement:

Fig. 47.—French Test for Permeability. (*See p.* 128.)

> The permeability of neat cement and mortars is expressed by the number of liters of water passed in one hour through a cubical block, 50 sq. cm. (7.75 sq. in.) on a face, the size used for compressive tests. The block is placed on its side, that is, with a face which has been against the mold uppermost; this surface is carefully cleaned and a glass tube 3 5 cm. (1.38 in.) in diameter, and 11

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*Commission des Méthodes d'Essai des Matériaux de Construction, 1894, Vol. I, p. 313.

†Proceedings American Society of Civil Engineers, August, 1903, p. 649.

cm. (4.33 in.) high is sealed vertically to it by means of neat cement, as shown in Fig. 47. For varying the pressure, a rubber pipe is attached to this tube, and its upper end connected with a reservoir. The height of pressure, according to the permeability of the mortar, may be 10 cm. (4 in.), 1 m. (3 ft. 3 in.) or 10 m. (33 ft.).

Before taking the test, the block is immersed in water for 48 hours, and remains in water during the test. The periods recommended are: 24 hours, 7 days, 28 days, 3 months, etc. The standard test is made at 28 days. Tests are made on three blocks, and an average taken of the two which most nearly agree.

Logically, we should suggest for the block to be used for testing permeability in this country, the size mentioned for compression, that is, a 2-inch cube. Further investigation is considered necessary, however, before adopting either the size or shape as a standard.

Since the publication of the above discussion, the authors have performed a series of tests on the relative permeability of concretes, as described on page 426, obtaining satisfactory relative results by cementing a short length of pipe to the surface of a solid block of concrete in a manner similar to that adopted by the French Commission.

YIELD TESTS OF PASTE AND MORTAR

The French Commission* recommend the testing of cement paste and mortar to determine the volume occupied. The yield or *rendement* is the volume of mortar obtained by gaging to any given consistency a unit of weight of cement or of a mixture of cement and sand in the selected proportions. One kilogram of cement, or of the required mixture of cement and sand, gaged to the given consistency, is introduced into a graduated cylindrical glass test tube about 6 cm. (2.37 in.) in diameter, with care to avoid imprisonment of air, and its volume is noted.

Another method, which they consider more accurate, is to allow the paste or mortar to harden and then determine the difference in weight in air and in water.

Mr. R. Feret in his report to the French Commission† on the production and density of mortars considers that sands should be submitted to a thorough test. He advises determining their granulometric composition, as described on page 142, the proportion of gravel (that is, of particles remaining on a sieve with holes of 50 mm. (0.19 in.) diameter, the mineralogical nature, and the form of the grains. Disregarding the yield test he would study the absolute volumes of the cement, the sand, the water,

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*Commission des Méthodes d'Essai des Matériaux de Construction, 1894, Vol. I, p. 307.

†Commission des Méthodes d'Essai des Matériaux de Construction, 1895, Vol. IV, p. 243.

and the voids in a unit volume of fresh mortar, and would estimate the cost per cubic meter of mortar made with different sands, and its strength under various conditions, as is discussed at length in the following chapter.

TEST OF RISE IN TEMPERATURE WHILE SETTING

The determination of the rise in temperature which takes place in a cement while setting has often been suggested as an indication of its quality, but the increase in temperature is due to so many causes that it is of slight value as a test of the cement.

Mr. Le Commandant Ribaucour* found that the temperature commenced to rise at the commencement of the setting, and the rise was generally higher with quick-setting cements.

Mr. J. E. Howard at the Watertown Arsenal† discovered that the temperature was largely dependent upon the size of the specimen, small cubes showing very slight increase. He therefore made a series of tests upon 12-inch cubes to determine the temperature acquired by different brands of cement and mortars during setting, and plotted his volumes in a series of curves. The curves for a first-class brand of American Port-

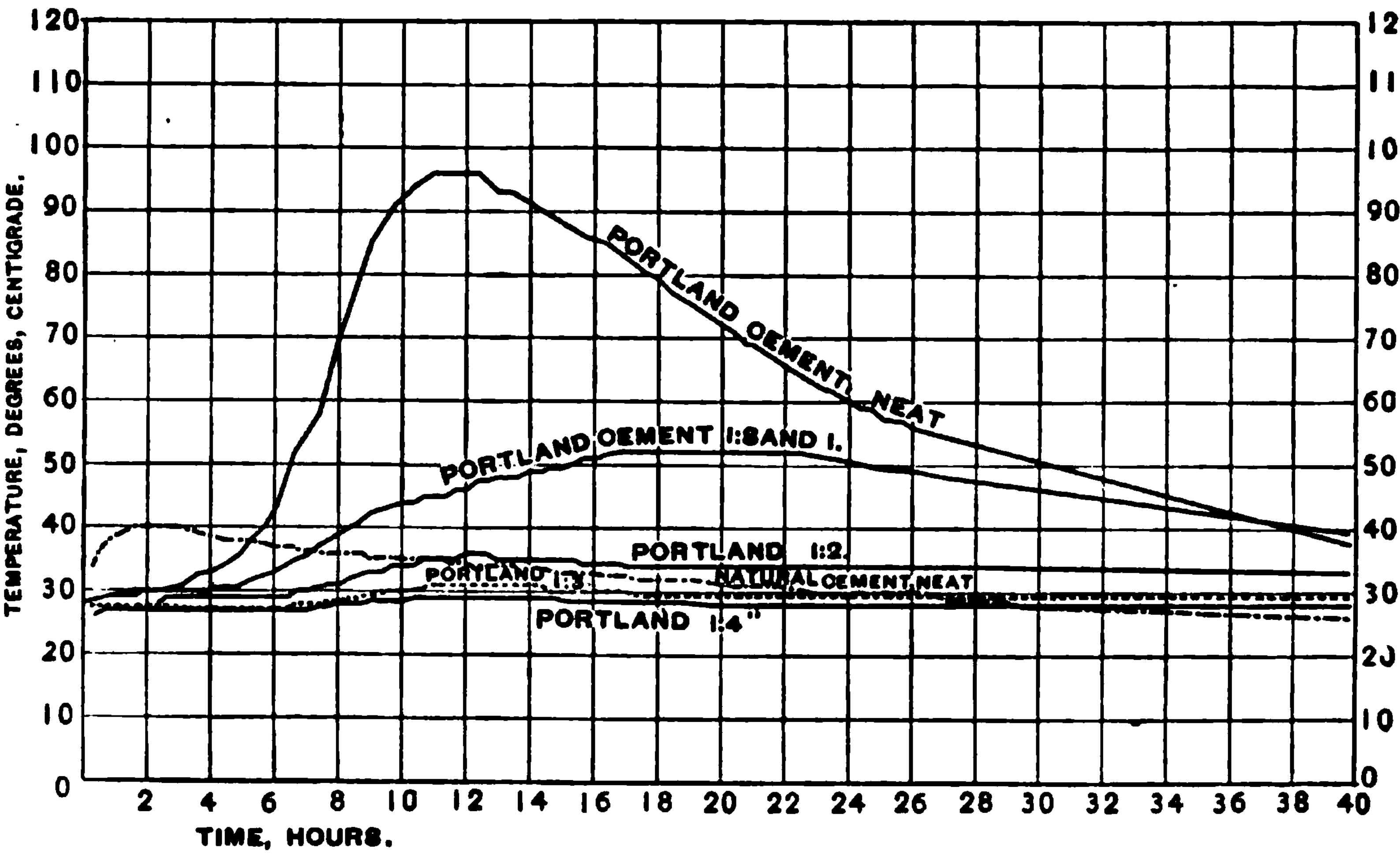

Fig. 48.—Rise in Temperature in 12-inch Cubes of Cement and Mortar. (Tests of Metals, U. S. A., 1901.) (*See p.* 130.)

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*Commission des Méthodes d'Essai des Matériaux de Construction, 1895, Vol. IV, p. 133.

†Tests of Metals, U. S A., 1901, p. 493.

land cement with and without sand, and for a typical Natural (Rosendale) cement, are shown in Fig. 48.

Mr. Howard found that while first-class American brands of neat Portland cement often reached a maximum temperature of 100° Cent. (212° Fahr.); the maximum temperature of the various brands of American Natural cement was generally from 35° to 40° Cent. (95° to 104° Fahr.), and was reached at a shorter time than the Portland cements. The rise in temperature of the German brands of Portland cements was in general less than that of the American Portlands.

The rise in temperature in Portland cement concrete is less than in neat Portland cement, but in the interior of a large mass like a dam may reach nearly 100° Fahrenheit.

CHAPTER IX

STRENGTH AND COMPOSITION OF CEMENT MORTARS

The following are the important conclusions in this chapter:

(1) The strength of a mortar depends primarily upon (*a*) percentage of cement in a unit volume, and (*b*) density. (See p. 133.)

(2) The strongest mortar for any given proportions, by weight, of cement to dry sand, is obtained from sand which with the given cement produces the smallest volume of plastic mortar. (See p. 148.)

(3) The best sand is invariably that which will produce the smallest volume of mortar of standard consistency when mixed with the given cement in the required proportions. (See pp. 133 and 149.)

(4) The density of a mortar is determined by calculating the absolute volumes of its ingredients. (See p. 139.)

(5) The qualities of different sands may be studied by screening each into three sizes and comparing their granulometric compositions with Feret's curves. (See p. 142.)

(6) Sharpness of the sand grains is of slight importance. (See p. 153.)

(7) Coarse sand produces stronger and usually more impervious mortar than fine sand. (See p. 146.)

(8) Fine sand requires more water than coarse sand to produce a mortar of like consistency, and consequently its mortar is less dense. (See p. 146.)

(9) Mixed sand, *i. e.*, sand containing fine and coarse grains, in mortars leaner than 1 : 2, usually produces stronger and more impervious mortars than coarse sand. (See p. 146.)

(10) Screenings from broken stone usually produce stronger mortars than sand because of their greater density. The relative value of screenings and sand may often be determined by comparing their densities or the densities of mortar made from them. (See pp. 148 and 151.)

(11) Mixtures of fine and coarse sand or of sand and screenings often produce better mortar than either material alone. (See p. 148.)

(12) The variation of the sand in different portions of the same bank may be utilized by requiring the contractor to mix two sizes without exact measurement, so that the material as delivered shall contain not less than

a definite percentage of sand coarse enough to be retained on a certain sieve. (See p. 148.)

(13) Impurities in sand, such as loam and clay, in small quantities, may strengthen a lean mortar, and weaken a rich mortar. (See p. 154.)

(14) Gaging with sea water does not affect the ultimate strength of mortars. (See p. 159.)

(15) The unit fiber stress in a cement or mortar beam is about the same for a prism 4 cm. (1.6 in.) on edge as for one 2 cm. (0.8 in.) on edge. (See p. 134.)

(16) The unit fiber stress in bending is about 1.89 times the unit tensile strength of briquettes of 5 sq. cm. (See p. 134.)

(17) The unit tensile strength of specimens decreases as the breaking area is enlarged. (See p. 134.)

(18) The unit compressive strength of similar specimens of cement or mortar is not greatly affected by their size. (See p. 134.)

Laws of Strength. There are two fundamental laws of strength which apply to mortars composed of the same cement with different proportions and sizes of sand.

(1) With the same aggregate,* the strongest and most impermeable mortar is that containing the largest percentage of cement in a given volume of the mortar.

(2) With the same percentage of cement in a given volume of mortar, the strongest, and usually the most impermeable, mortar is that which has the greatest density,† that is, which in a unit volume has the largest percentage of solid materials.

The first of these rules is understood by ordinary users of cement, but the second rule states a fact which is appreciated only by experts.

The value of a first-class cement is universally recognized, the effects of impurities have been studied in various ways, and the variations in strength of mortars made from different sands or broken stone screenings have been recorded, but the fundamental law of the relation of the density of a mortar to its strength, — a function nearly as important as the quality of the cement itself and explaining many of the seemingly paradoxical results of tests with different aggregates and different proportions of water, — is but vaguely comprehended by the majority of experimenters and most of the users of cement.

The importance of this subject claims for it a full investigation, and its study is taken up on page 134. The application of these laws to concrete is discussed in Chapter XIII.

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*The word *aggregate* is defined on page 1.

†The meaning of *density* may be understood by referring to the figures on pp. 172 and 173.

STRENGTH OF SIMILAR MORTARS SUBJECTED TO DIFFERENT TESTS

Mr. René Feret, Chief of the Laboratory of Bridges and Roads at Boulogne-sur-Mer, France, has made very extended tests of strength of mortars, studying his results scientifically, and in many cases formulating laws and formulas applicable to different conditions. The tests of one series in particular are of so wide a range in character and in proportions used that the authors have converted the values into English units, and reproduce the table in full on pages 136 and 137.

After plotting the strengths in various ways, Mr. Feret reaches conclusions which may be summed up as follows:

(*a*) The unit fiber stress for prisms 4 centimeters (1.6 in.) on an edge is about the same as for prisms 2 centimeters (0.8 in.) on edge.

(*b*) The tensile strength per square centimeter of prisms having a breaking area of 16 square centimeters (the strength of which he found to be similar to that of briquettes of the same section) is about two-thirds the strength per square centimeter of the normal briquettes which have an area of 5 square centimeters. This difference is attributed partly to the lack of homogeneity of the specimens, especially on their surfaces, but principally to the unequal distribution of the stress on the area of the section.

(*c*) Resistance to flexion, that is, the unit fiber stress in bending, is about 1.89 times the tensile strength per unit of area of briquettes of 5 square centimeters.

(*d*) The form and dimensions of the specimen do not greatly influence the strength per unit of area in compression when the height and width of the block are approximately equal.

(*e*) Resistances to flexion and tension are proportional to each other, and resistances to compression, shearing, and punching are proportional to one another, but there is no constant relation between the resistance to compression and the resistance to tension or flexion.

THE RELATION OF DENSITY TO STRENGTH

In the same paper from which we have quoted, Mr. Feret treats of the density and elementary volumetric composition of mortars, using in his studies the results given in the table just described. He calls particular attention to the fact that the properties of hydraulic mortar, such as durability, permeability, porosity, and ability to resist the decomposing action of sea water, depend not only upon the quality of the cement, but "in a measure greater than is generally believed, upon the granular physical composition of the mortars, that is to say, upon the dimensions and relative positions of the different elements entering into their composition."

The density (*compacité*) of a mortar is represented by the total volume of the solid particles, — exclusive of the water and the voids, — entering into a unit volume of mortar.*

The "elementary volumes" in a unit volume of fresh mortar consist of the absolute volumes of the cement, sand, water, and voids, each expressed in the form of a decimal. To illustrate, the "elementary volumetric composition" of the mortar in Item 8 of the table on page 136, which is mixed in proportions by weight of one part cement to 1⅘ parts of natural sand, is:

Cement $(c) = 0.226$
Sand $(s) = 0.499$
Water $(w) = 0.234$
Air voids $(v) = 0.041$

Total volume $= 1.000$

Expressing this in more familiar terms, 22.6% of the unit volume of the given mortar consists of solid particles of cement, 49.9% of particles of sand, 23.4% of water, and the remaining 4.1% of air voids.

The porosity, represented by the sum of the water and air voids, is 27.5%. The term *voids* is often employed to represent the porosity, that is, the sum of the air and water.

It is obvious that

$$c + s + w + v = 1;$$

also that

$$v = 1 - (c + s + w),$$

which is equivalent to the statement that the entrained air in any volume of fresh mortar is equal to the measured volume of the mortar minus the space occupied by the cement, sand, and water.

Method of Determining Density. The density of the mortar considered above is $c + s$, or, $0.226 + 0.499 = 0.725$ as given in column (11) of the table.

A thorough understanding of the use of these symbols is essential to the study of strength of concrete and mortar, for, as will be shown further on, practical tests of strength are of small value unless the density and exact mechanical composition of the specimens are clearly defined.

The method adopted by the authors of obtaining the density and vol-

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*If the word density is applied to sand alone, it means the proportion of the measured volume of the sand, which is occupied by the solid sand grains; a sand, for example, having under certain conditions 40% voids, would have a density of $1.00 - 0.40 = 0.60$.

Strength and Composition of Portland Cement Mortars.

By R. Feret (*See p.* 134.)

		(1)	(2)	(3)	(4)	(5)	(6)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)
D	(21)	1	12.3	75	167	3178	1.49	0.0076	114	141		65	160	200	130	156	67	160
	(22)	1	5.8	148	168.5	3230	2.38	0.0269	247	238	60	122	600	550	480	370	126	540
	(23)	1	3.5	225	170	3262	3.32	0.0529	408	415	119	208	1080	1290	1320	768	214	1230
	(24)	1	2.4	299	172	3318	4.26	0.0818	576	576	196	296	1790	1910	2120	1410	302	1940
	(25)	1	1.8	363	175	3367	5.09	0.1082	703	693	233	354	2380	2970	3170	2130	364	2840
	(26)	1	1.3	429	179	3412	5.98	0.1376	814	828	233	439	3470	3570	4100	2570	436	3710
	(27)	1	1.0	500	183.5	3458	6.92	0.1681	980	963	302	502	4750	4410	5830	2750	510	5000
	(28)	1	0.7	573	190	3510	7.90	0.1998	1100	1120	373	549	5750	5450	6070	3070	574	5760
	(29)	1	0.5	659	197	3495	8.92	0.2171	1210	1280	371	623	6630	5970	6900	3570	647	6500
	(30)	1	0.3	771	208	3545	10.39	0.2530	1280	1370	401	671	7400	6830	7110	(4120)	691	>7110
M′	(31)	1	5.0	167	123	3650		0.0676	613	623	239	330	2120	2350	2570	1720	328	2350
	(32)	1	3.0	250	136	3642		0.1056	842	808	304	475	3640	3870	4510	3100	450	4010
	(33)	1	2.0	333	148	3645		0.1429	990	981	336	513	4640	4860	4920	3070	518	4810
N′	(34)	1	3.0	250	91	3620		0.1246	862		293	455	3600	3670			456	3640
C	(35)	1	0	1000	245	3552		0.2851	1280	1510	385	624	8720	7350	>7110	3680	698	8040

NOTE.—All are plastic mortars except N′.

EXPLANATION OF COLUMNS.

Col. (6) based on price and weight of given sand, on cement at 50 francs per "tonne" ($1.66 per bbl.), and on labor at 3 francs per cubic meter (44 cts. per cu. yd.) of mortar.

Cols. (7) to (12) are discussed on page 135.

Col.	Number of specimens of each mortar.	Size of specimens, centimeters.	Remarks.
(13)	15 prisms	4 x 4 x 16	Supports 10 cm. apart
(14)	15 "	2 x 2 x 13	" 10 " "
(15)	15 "	4 x 4 x 8±	Halves of broken prisms
(16)	25 briquettes	5 sq. cm. section	French standard briquettes
(17)	5 cubes	50 sq. cm. face	
(18)	15 prisms	4 x 4 x 8±	Halves of broken prisms
(19)	30 "	2 x 2 x 3±	Quarters of " "
(20)	15 "	2 x 2 x 6±	Halves " " "

(21) Average of cols. (14), (15) and (17) by formula

$$T = \frac{\left(\frac{F_{16} + F_4}{2}\right) + T_8}{1.89 + 1}$$

(22) Average of cols. (18), (19), (20).

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†Granulometric composition is defined on p. 142.

umetric composition of a mortar,* gives opportunity to study different aggregates and proportions as well as the effect of variable quantities of water upon the same dry materials. It is applicable also to concrete experiments. For mortar experiments glass tubes, at least 3 inches in diameter, or deep molds may be used for measuring the volumes. For concrete a piece of 6-inch or 8-inch pipe is convenient. The volume of mortar and concrete of dry consistency will measure the same after setting as when green, but wet mixtures must be measured before setting, and again after they have become sufficiently hard to expel the surplus water. The measurement before setting is necessary in order to calculate the volume of air-bubbles entrained in the wet mortar or concrete. The volume after setting, or partially setting, however, is the only one of real importance for studying the characteristics of strength, permeability, and cost. The sand is dried, or its moisture is determined by weighing and drying a sample of it. If stone of a porous nature is used the pores of its particles should be filled with water, but there should be no perceptible moisture on their surfaces. The quantities of dry materials for a single tube or mold are weighed in the required proportions, mixed with a known weight of water, and placed compactly in the mold, whose lateral dimensions have been exactly measured so that the volume of mortar in it may be obtained by measuring down from the top. The exact space occupied by the particles of each of the solid materials and by the water is calculated, if the metric system is employed, by dividing their total weight by the specific gravity of each, or, if English units are used, by dividing the weight times 1728 (the number of cubic inches in a cubic foot) by the specific gravity multiplied by the weight of a cubic foot of water. After partially setting, the exact depth of the mortar in the mold is measured and its volume calculated. The percentage of each of the dry materials, which really determines the density, — which is represented by the sum of the absolute volumes of the dry material, — is found by dividing the absolute volume of each material by the total volume of the set mortar or concrete.

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*The French Commission determine the "yield" of a mortar (see p. 129) by measuring its volume green, that is, just after introduction into the molds, when an excess of water may affect the volume, and thus give misleading results with very wet mixtures.

In his Report to the French Commission, 1895, Vol. IV, p. 243, Mr. Feret also measures the mortar wet, but he employs a vessel of known capacity, — a cylindrical measure whose height and interior diameter are each about 8 centimeters, — and uses only a portion of the mortar which he mixes, calculating his percentages bv ratio of the weight of mortar made to the weight of mortar introduced into the measure to fill it exactly. This method eliminates inaccuracies in measuring the level of the surface.

The specific gravity of cement which has been stored for a short time may be taken at 3.10, and the specific gravity of dry sand at 2.65.

The following example from the authors' note book illustrates the method of finding the density when the measurements are in English weights and measures:

Example: — Find density of a mortar composed of Newburyport sand and Portland cement in proportions 1: 2 by weight.

Solution: — For the mold used, it was estimated that 8 lb. cement and 16 lb. dry sand would be required. Gaging these with 3 lb. 12.6 oz. (3.79 lb.) of water, the quantity necessary for the desired consistency, the volume of the mortar was found by measurement to be 348 cu. in. when green, and 336 cu. in. after setting and pouring off the surplus water. The absolute volumes are expressed below, first in cubic inches and finally in terms of the density ($c + s$), of the set mortar.

$$\text{Cement} = \frac{8 \times 1728}{3.1 \times 62.3} = 71.6 \text{ cu. in.}$$

$$\text{Sand} = \frac{16 \times 1728}{2.65 \times 62.3} = 167.4 \text{ cu. in.}$$

$$\text{Water} = \frac{3.79 \times 1728}{62.3} = 105.1 \text{ cu. in.}$$

Absolute volume cement, sand and water,	344 cu. in.
Measured volume green mortar,	348 cu. in.
Volume of entrained air,	4 cu. in.
Percentage of entrained air,	1.2%

$$\text{Density of set mortar, } c + s = \frac{71.6}{336} + \frac{167.4}{336} = 0.213 + 0.498 = 0.711$$

Feret's Formula for Strength. For studying the relation of absolute volumes to strength, let

P = compressive strength of the mortar.
K = a constant which differs for different cements and at different ages of the same mortar.
c = absolute volume of cement.
s = absolute volume of sand.
w = absolute volume of water voids.
v = absolute volume of air voids.

The value of determining the density of mortars is made evident by the following law of Mr. Feret:*

"For any series of plastic mortars made with the same binding material

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*Bulletin de la Société d'Encouragement pour l'Industrie Nationale, 1897, Vol. II, p. 1604.

and inert sands, the resistance to compression after the same length of set, under identical conditions, is solely a function of the ratio $\frac{c}{w + v}$ or $\frac{c}{1 - (c + s)}$, whatever be the nature and size of the sand and the proportions of the elements, — cement, inert sand and water, — of which each is composed."

It follows from this law, as Mr. Feret says, that the strength of any

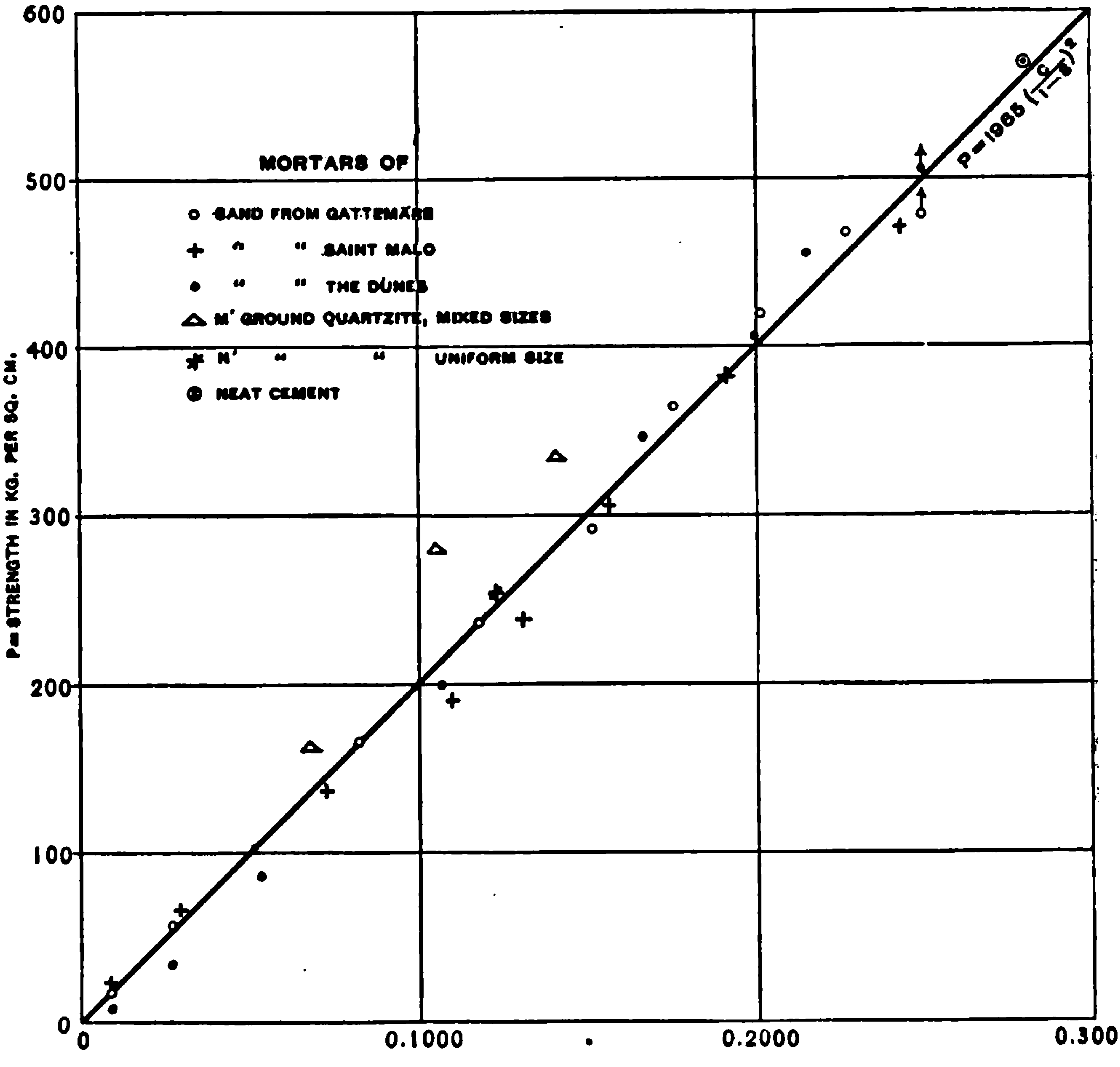

FIG. 49. — Derivation of Feret's Formula for Strength. (*See p.* 141.)
(Bulletin de la Société d'Encouragement pour l'Industrie Nationale — 1897.)

mortar increases with the absolute volume of the cement (c) in a unit volume of fresh mortar, and also with the density ($c + s$), whatever may be the relative volumes filled with water and air.

From very numerous experiments such as those tabulated on page 136 Mr. Feret evolves the approximate formula

$$P = K\left(\frac{c}{1 - s}\right)^2 \tag{1}$$

By suitably changing the value of K the formula may be adapted to either the English or the metric system of measurement.

As a proof of this formula Mr. Feret plots on a diagram, shown in Fig. 49, values of $\left(\frac{c}{1 - s}\right)^2$ from column (12) in the table on pages 136 and 137 for abscissas, and the average compressive strengths of the various mortars, from column (22), for ordinates. Since, in formula (1), K is equal to P divided by the square of the quantity in brackets, the value of K is the tangent of the straight line passing through the points. In Fig. 49

K = 1965, if the strength is in kg. per sq. cm.

or

K = 28 000, if the strength is in lb. per sq. in.

This particular value is applicable only to the cement used by Mr. Feret in his experiments and to specimens at the age of five months, but the principles involved are of general application.

The most practical application of this formula is in the determination of the relative compressive strengths of various mortars made from the same cement, with sand in differing proportions and of different compositions. Mr. Feret calls attention also to its possible use in laboratory experiments and specifications. A cement, for example, may be required to furnish, when mixed with any sand, a definite value of K, since the value of K is independent of the choice of the sand and of the composition of the mortar.

Experiments by the authors tend to show that the formula does not apply strictly to specimens of different consistency, but that the general law of the increase of strength with the density is applicable except in extreme cases. The formula is inapplicable to tensile tests, although here, too, the general principle appears to hold good.

This subject as related to concrete is discussed on pages 237 to 244

GRANULOMETRIC COMPOSITION OF SAND

Feret's Three-Screen Method of Analyzing Sand.

The determination of the physical characteristics of the sand, which, mixed with a cement, will produce the densest mortar, has been the object

of a large number of experiments by Mr. Feret, which are recorded in Annales des Ponts et Chaussées, 1892. In America Mr. William B. Fuller has extended the researches, by a different method, to the investigation of the properties of concrete. Mr. Fuller's mechanical analysis of sand and stone is discussed by him in Chapter XI, and the results of his experiments are tabulated on page 258.

Mr. Feret, in studying any sand, separates it by screening into three sizes. He then recombines these three sizes in varying proportions, so as to obtain results which are applicable to any natural or artificially mixed sand. He distinguishes sand from gravel as consisting of grains which will pass through a screen having circular holes of 5 millimeters diameter (0.20 in.). The three sizes of sand he then calls G, M, and F, representing, respectively, the large (*gros*), medium (*moyens*), and fine (*fins*) particles as defined by sifting through metallic sieves with circular holes, or wire cloth of definite mesh, as follows:

Large grains, G, passing circular holes	5 mm.	(0.20 in.)	diameter.
Retained by circular holes	2 mm.	(0.079 in.)	"
Medium grains, M, passing circular holes	2 mm.	(0.079 in.)	"
Retained by circular holes	0.5 mm.	(0.020 in.)	"
Fine grains, F, passing circular holes	0.5 mm.	(0.020 in.)	"

These sizes, Mr. Feret states, are nearly equivalent to sand screened through sieves of wire cloth as follows:

Large grains, G, passing screen of	4	meshes	per sq. cm.	(5	meshes	per linear	inch.)
Retained on "	36	"	"	(15	"	"	")
Medium grains, M, passing "	36	"	"	(15	"	"	")
Retained on a "	324	"	"	(46	"	"	")
Fine grains, F, passing "	324	"	"	(46	"	"	")

Sometimes, for experimental purposes, he divides each of the sands, G, M, and F, into three intermediate sizes.

The granulometric composition of any sand is represented by its relative proportions, expressed either in weights or absolute volumes, of G, M, and F. For example, a sand containing by weight 50% of the largest grains, 30% of the medium, and 20% of the fine grains, has a granulometric composition of $g = 0.50$, $m = 0.30$, $f = 0.20$.

The granulometric composition of a sand which has been mechanically analyzed, and plotted on a diagram similar to that shown on page 194, may be ascertained readily by drawing three ordinates corresponding respectively to screens of 5, 15, and 46 meshes per linear inch, and determining by the length or the difference in length of these ordinates the proportions which pass and which are retained by the screens of these three meshes. These three proportions or percentages represent the granulometric com-

position. An illustration of this method of transforming mechanical analysis to granulometric composition is shown in Fig. 57 on page 150.

Feret's Triangles. To simplify the tabulation of results, and arrange them so that they may be understood at a glance, Mr. Feret has used a graphical arrangement which is exceedingly ingenious. In nearly all his writings we find little triangles with the apexes labeled G, M, and F. Curves or contours in these triangles, representing the various properties of the sands or mortars, are based on a system of three instead of two

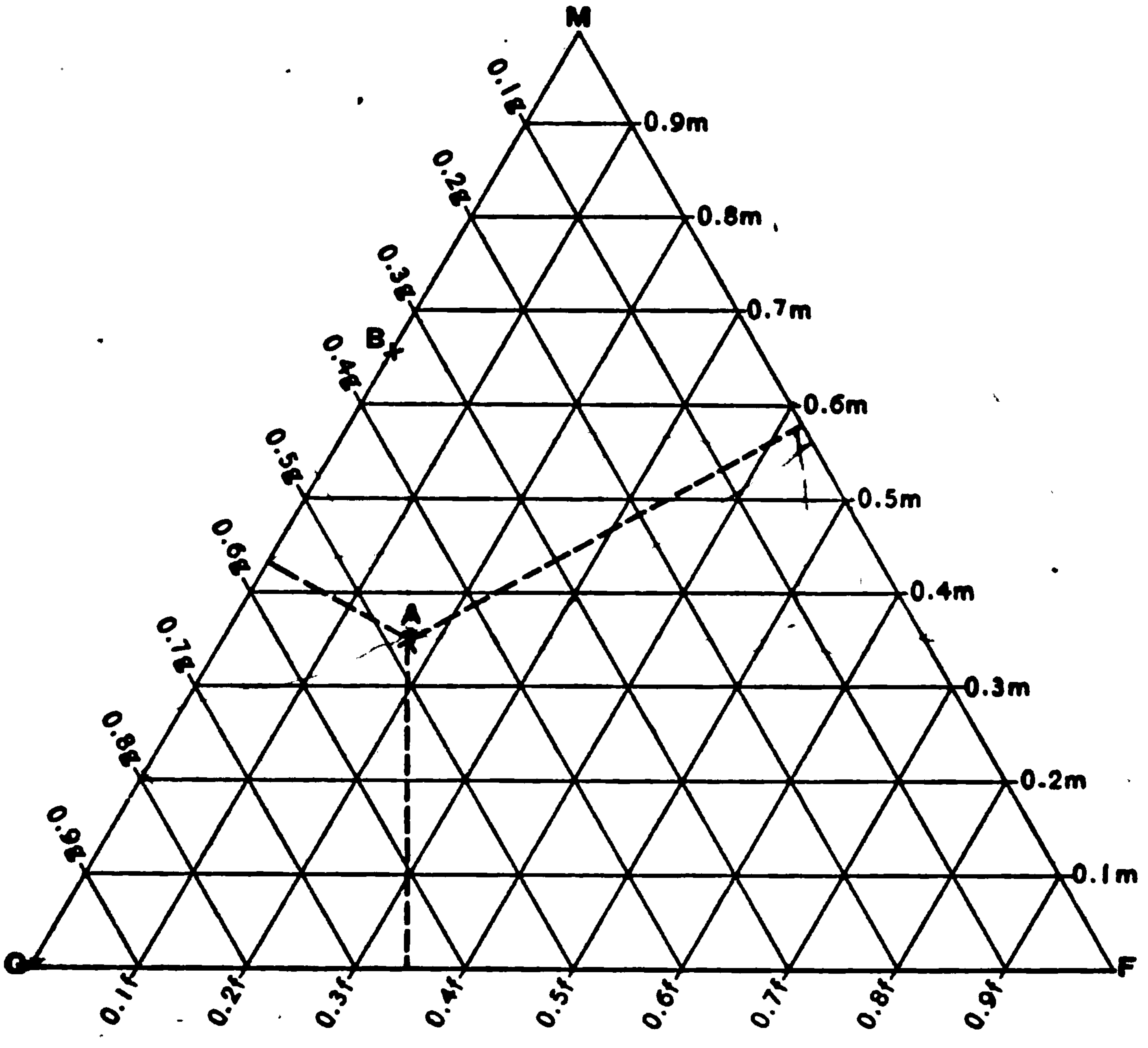

Fig. 50.—Feret's Three-Screen Method of Analyzing Sand. (*See p.* 143.)

co-ordinates, that is, each curve is the loci of points measured from 3 axes placed at angles of 60° with each other. A full discussion of the theory of this is given in his paper "Sur la Compacité des Mortiers Hydrauliques" in Annales des Ponts et Chaussées, 1892, II, but the principles may be understood by reference to Fig. 50. The apexes of the triangle are labeled G, M, and F, corresponding to the three sizes of sand described on page 142. The granulometric composition of any sand is plotted as a single point in this triangle. The proportion of each of the three sizes in the sand is represented by its perpendicular distance from the side opposite each apex.

For example, exactly at the apex G, the granulometric composition is g = 1.00, m = 0, f = 0. A sand represented by the point "*A*" in the triangle has for its granulometric composition, g = 0.48, m = 0.35, f = 0.17. Sand, *B*, whose point is on the line G M is a mixture of G and M with no fine particles. It can be readily proved by geometry that if the altitude of the triangle is 1.00, the sum of the three perpendicular distances from any given point in the triangle to the three sides equals 1.00. Also, that any combination of G, M, and F is contained in the triangle or else on one of its sides. To use Mr. Feret's language, "any sand will be represented by a point in the triangle and by one alone, and, reciprocally, one granulometric composition of sand, and only one, will correspond to a given point on the interior or sides of the triangle." If the altitude of the triangle is considered 1.00, any point, *A*, in the triangle is readily plotted by locating it at perpendicular distances from each of the three sides corresponding to each component of its granulometric composition. For example, suppose that the granulometric composition of a sand, *A*, is g = 0.48, m = 0.35, f = 0.17. As the apex G represents a sand containing only coarse grains, and the line opposite to it, M F, all sands containing no coarse grains, the locus of a sand containing coarse grains (g = 0.48) will lie somewhere upon a line parallel to M F and at a distance 0.48 from M F. By similar reasoning it will also lie on a line parallel to G F and at a distance 0.35 from it. The intersection of these two lines is the locus of the sand *A*, and it will be seen that this intersection is at a perpendicular distance of 0.17 from the line M G (the side opposite F), which checks the plotting, since f = 0.17.

n

F

FIG. 51.—Absolute Volumes of Sand per Unit Volume of Sand not Shaken. (*See p.* 146.)

FIG. 52.—Absolute Volumes of Sand per Unit Volume of Sand Shaken to Refusal. (*See p.* 146.)

For comparing a special property of different sands, or of mortars com-

posed of different sands, each sand employed in the tests is plotted and labeled with its value, — which may be in units of strength, weight, or volume, — and "contour lines" are sketched in by the eye, as one would draw contours from elevations on a topographical drawing.

Any point on the same contour line represents a sand made up of the different sizes, G, M, and F, in proportions corresponding to its perpendicular distances from the sides opposite each apex, but having the same strength, weight, volume, humidity, or whatever special function may be represented, as every other point on the same line.

FIG. 53.—Absolute Volumes of Solid Materials (c+s) per Unit Volume of Fresh Mortar in Proportions 1:3 (by Weight). (*See p.* 146.)

FIG. 54.—Compressive Strength in Pounds per Square Inch of 1:3 (by Weight) Mortars with Different Mixtures of Sand, after 9 Months in Air and 3 Months in Sea Water. (*See p.* 147.)

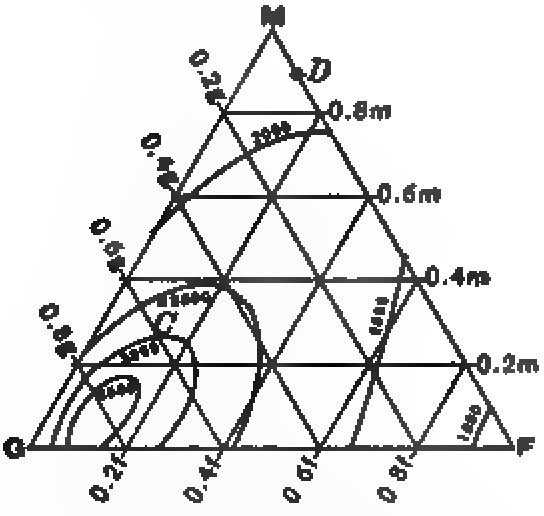

FIG. 55.—Compressive Strength in Pounds per Square Inch of Mortars with Various Mixtures of Sand, after One Year in Fresh Water. Proportions 100 lb. Portland Cement to 3.2 cu. ft. Mixed Sand. (*See p.* 147.)

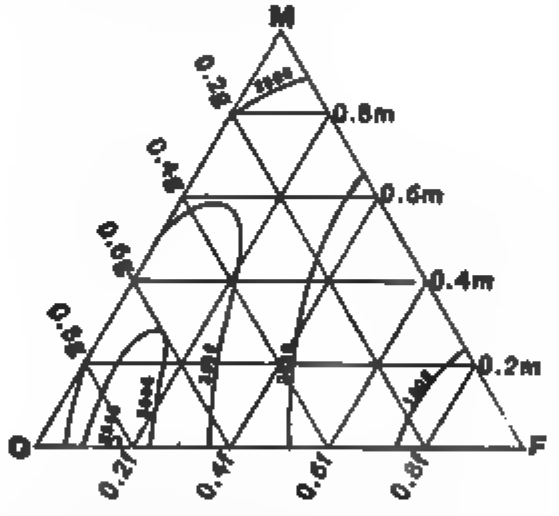

FIG. 56.—Compressive Strength in Pounds per Square Inch of Mortars with Various Mixtures of Sand, after One Year in Air. Proportions 100 lb. Portland Cement to 3.2 cu. ft. Mixed Sand. (*See p.* 147.)

Figs. 51 and 52, page 144, illustrate the use of the triangle for showing the volumes of sands composed of different sizes of grains. Any sand, for example, whose granulometric composition is represented by any point on the contour line labeled 0.575, in Fig. 51, has, when measured loose, 0.575 of its volume, or 57½%, of absolutely solid matter, or, taking the complement, 42½% of voids. In Fig. 51 it will be seen that the greatest solid volume of loose sand is obtained by mixing G and F in proportions 60% G and 40% F by weight. The amount of solid matter in this mixture of maximum density is 0.61 of the unit volume; in other words, the sand contains 39% voids. By interpolating between the contour lines we may see that a sand consisting of equal parts of the three sizes, which would be represented by a point at the geometrical center of the triangle, has about 0.597 solid matter, or 40.3% voids. In sands shaken to refusal, Fig. 52, the mixture of maximum density consists of sands G and F alone, in proportions about 55% G and 45% F, and the total solid matter, that is, the absolute volume of sand, in a unit volume of the shaken sand of maximum density, is 0.798, corresponding to 20.2% voids.

EFFECT OF SIZE OF SAND UPON THE STRENGTH OF MORTAR

As a matter of fact, the actual size of a sand, that is, the size of its grains, is subordinate, in its influence upon the strength and other qualities of a mortar, to the density of the mortar produced from it. One naturally would suppose that the densest sand, that is, the sand which contains, when dry, the fewest voids, when mixed with a given proportion of cement, would make, inevitably, the densest and therefore the strongest mortar. Such, however, is not necessarily the case, for the addition of both the cement and water change the mechanical composition. A mixture of fine sand and cement, for example, requires a larger percentage of water in gaging than a mixture of coarse sand and the same cement. The total volume of a mortar of plastic consistency is affected by the quantity of water used, as well as by the volumes of the dry materials. Hence, a mortar consisting of fine sand and cement will be less dense than one of coarse sand and the same cement, even though the fine and coarse sands, when weighed or measured dry, each contain the same proportions of solid matter and voids.

Fine sand has more grains in a unit measure and therefore a greater number of points of contact of the grains. The water forms a film (see Fig. 63, p. 175,) and separates the grains by surface tension.

The fact is graphically illustrated in Feret's triangle, Fig. 53, page 145,

in which the contour lines show the combined absolute volumes of the cement and sand in 1 : 3 mortar (proportioned by weight) made from sand of various compositions. It will be noticed that the point of maximum absolute volume, which is labeled 0.734, is much farther to the left than in Figs. 51 and 52, showing that for a mortar of maximum density, a sand is required containing more large particles, G, in proportion to the fine particles, F, than for maximum density with the same sand in its dry state.

From such experiments Mr. Feret* derives the law that:

> The plastic mortars, which, per unit of volume, contain the greatest absolute volume of solid materials (c + s), are those in which there are no medium grains, and in which coarse grains are found in a proportion double to that of fine grains, cement included.

Figs. 54, 55, and 56, page 145, show the strength in compression, converted to pounds per square inch, of mortars made from various mixtures of the three sizes of sand.

Comparing these with Fig. 53 it will be seen that the curves of strength follow the same general direction as the curves of density. This is in conformity with the general laws stated at the commencement of the chapter and with the principles upon which Feret's formula (page 141) is based.

There is one point which must be noticed when studying these and other similar triangles of Feret, namely, that his results, as shown by the curves on his triangles, apply exactly only to sands and cements, and not to mixtures of sand and coarse stone. In all the triangles, sands for maximum density are composed of a mixture of fine and coarse grains with no medium grains. It is shown on page 172 that a denser mixture can be obtained with stone and sand and cement, that is, with three sizes of materials, than with sand and cement, and it is consequently probable that Feret could have obtained greater densities by making the size of G larger (that is, employing for G gravel or broken stone) and the size of F smaller, and that with this arrangement a portion of the medium grains would have been absolutely necessary to obtain the maximum density. In this connection, however, it must be remembered that Feret's experiments were intended to cover, as far as possible, practical combinations of sizes of sand for mortar. It is noticeable, even with the sizes of sand which he uses, that the curves in Fig. 53 run sharply upward, and that mortars from mixtures of three sizes of sand are therefore very nearly as dense and strong as those made from two sizes. Furthermore, when the three sizes

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*Annales des Ponts et Chaussées, 1896, II, p. 182.

G, M, and F are mixed together, a graded mixture is formed in which there are particles ranging from 0.2 inch down to fine dust.

PRACTICAL APPLICATIONS OF THE LAWS OF DENSITY

It is probable that many who read this chapter will question the practical use of it all. Sand from the same bank usually varies largely in different places, and even when sands of a uniform character are to be obtained, it is considered impracticable to mix two or more sizes on account of the expense involved. In other cases, only one quality of sand is obtainable, and consequently there is no opportunity for choice.

In answer to such critics, we outline below several conditions under which the investigation of the physical properties of the sand is not only interesting but essential from the standpoint either of quality or of maximum economy.

(*a*) The variation of the sand in different portions of the same bank may be utilized by requiring the contractor to mix two sizes without exact measurement, so that the material as delivered shall contain not less than a certain percentage of sand coarse enough to be retained on a certain sieve.

(*b*) If two sands are available, a study of their physical characteristics will determine which is better suited to the work in hand as *the sand which produces the smallest volume of plastic mortar, when mixed with cement in the required proportions by dry weight, furnishes the strongest and least permeable mortar.*

(*c*) A good sand brought from a distance at a high price may be more economical than a poor sand from a neighboring bank.

(*d*) The relative value of crusher dust or of sand in a given locality may be determined by comparing their densities or the densities of mortars made from them.

(*e*) Frequently, a mixture of a fine and coarse sand, or of sand and crusher dust, proportioned according to their relative granulometric compositions or analyses, may be shown to produce a better mortar than either material alone.

(*f*) To produce impermeable mortar or concrete, it may be economical to screen a mixed gravelly sand into different sizes, and remix these in proportions which will produce a mortar of greater density.

(*g*) The value of "sand cements" for use in mortar and concrete under certain conditions may be made evident.

All these points may be determined without resorting to the expensive,

tedious, and sometimes misleading tensile tests of sand mortars, except as an auxiliary requirement or for checking the established conclusions.

The use of mixed sand, as described in (*a*), was adopted by Mr. Thomas F. Richardson, Engineer, for the 1: 2 Natural cement mortar employed in the stone masonry of the Wachusett dam of the Massachusetts Metropolitan Water Works, after an exhaustive study of the comparative tensile strength and permeability of mortars made with different sands. He required the contractors to furnish sand so coarse that at least 50% would be retained on a sieve having 30 meshes per linear inch. The sand was excavated by scrapers, and the condition was readily complied with, whenever the sand in one section was shown by samples to be running too fine, by taking alternate scraper loads of coarse sand from another place in the bank.

Comparative Tests of Different Sands. One of the most important applications of the laws of density is in the comparison of different sands. Void determinations of sand are valueless because of variations in moisture and compactness, but if equal dry weights of each of the sands to be compared are mixed with the same cement in the proportions required on the work, and then gaged to plastic consistency as described on page 138, the best sand is that which produces the smallest volume of mortar.

CONVERSION OF MECHANICAL ANALYSIS TO GRANULOMETRIC COMPOSITION

As an illustration of methods of contrasting two different sands and of making practical use of Feret's researches, we may compare tests made by Mr. R. L. Humphrey* in connection with the construction of the Pennsylvania Avenue Subway, Philadelphia. He found the tensile strength at the age of one year, of 1: 3 mortar made with sand screened from gravel, to be about 50% stronger than that made with sand dredged from the Delaware River. The mechanical analyses† of the two sands are plotted by the authors in Fig. 57, page 150, from tables presented by Mr. Humphrey.

To transform these mechanical analysis curves to Feret's granulometric composition, we may draw on the diagram, ordinates corresponding to the sizes of sieves used by him, namely, No. 5, No. 15, and No. 46. (See p. 142.) From inspection of the curve it is evident that the granulometric composition of the gravel sand is $g = 0.56$, $m = 0.35$, $f = 0.09$, and of the river sand is $g = 0.00$, $m = 0.89$, $f = 0.11$. Plotting these granulometric compositions

*Transactions American Society of Civil Engineers, Vol. XLVIII, p. 558.

†Mechanical Analysis Curves are described in Chapter XI, page 190.

as *C* and *D* on Feret's triangle, Fig. 55, and interpolating between contours, we find the relative compressive strengths of mortars made from the two sands to be, after one year in fresh water, about as 1775 is to 2550, or as 1 : 1.44, while Mr. Humphrey's ratio of tensile strength for the two mortars at the age of one year is as 304 is to 470, or as 1 : 1.53. These ratios are remarkably similar when the differences in conditions are considered.

Numerous tests have been made in America* in proof of the general law

FIG. 57.—Conversion of Mechanical Analysis to Granulometric Composition. (*See. p.* 149.)

that coarse sands are stronger than fine. Many experimenters have seemed to reach the result that coarse sand is stronger than mixed sand. In certain cases this is undoubtedly true, because of mixing the different sizes in wrong proportions, or because the mortar of coarse sand contains so large a proportion of cement that the voids are completely filled and the addition of fine sand increases, instead of decreasing, the density. Mortar, for example, as rich as 1 : 2 (*i.e.*, one part cement to two parts sand) of coarse sand is as strong, and less permeable, than mortar of similar proportions made of almost any mixed sands, but with leaner mortars, a small admixture of from 20% to 25% of fine sand improves it. Natural sand which in appearance is very coarse, almost invariably has a small percentage of very fine particles which, with the fine grains of cement, may assist, in the leaner

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*E. S. Wheeler in Report Chief of Engineers, U. S. A., 1895, p. 3013, A. S. Cooper in Journal Franklin Institute, Vol. CXL, p. 326, Ira O. Baker in Journal Western Society of Engineers, Vol. I, p. 73.

mixture, in producing a dense mortar. The mechanical analysis curves of sand shown in Fig. 72, on page 194, are an illustration of the fine matter contained in all bank sands.

EFFECT OF QUANTITY OF WATER UPON THE STRENGTH OF MORTARS

Fine sands require in gaging a larger percentage of water than coarse sands, in order to produce a mortar of the same consistency. This, as discussed on page 146, exerts an indirect influence upon the strength.

The influence of different percentages of water upon the same cement and aggregate is largely physical, although a deficiency may affect the permanent strength of a mortar, while an excess may for reasons given on page 271 injure the cement by dissolving a portion of it.

The effect of different proportions of water upon the ultimate strength (as suggested on p. 141) depends chiefly upon the density of the resulting mortar; the consistency which produces with a given weight of the same materials, the smallest volume, after setting, of Portland cement paste or mortar, gives the highest strength. Dry mixed mortars usually test higher than wet, — especially at short periods, as they set and harden more rapidly, — because they can be more densely compacted, but more uniform results in practice as well as in experiment, can be attained with plastic mixtures.

Tests by Mr. E. S. Larned,* a portion of which are shown in the table on page 152, illustrate the practical effect of different proportions of water upon the strength of neat cement pastes at various periods. It is noticeable that although the Natural cement mixed very wet finally attains a high strength, its very low strength up to 28 days shows the inadvisability of mixing Natural cement with an excess of water.

SAND VS. BROKEN STONE SCREENINGS

The relative strength of mortars made from sand and from screenings of broken stone or crusher dust has occasioned much discussion and dispute. It is probably dependent chiefly upon the relative density of the different mortars. Usually, a mortar from screenings will show higher tests, while occasionally mortar from sand will be superior, because of the difference in size or of the relative sizes of the particles or grains composing the two materials.

*Proceedings American Society for Testing Materials, Vol. III, 1903, p. 401.

Table Showing Strength of Cements Mixed Neat with Different Proportions of Water.

By Edward S. Larned. (*See p.* 151.)

Cement brand	Water per cent	Sieve test residue on			Wire minutes		Tensile strength					
		No. 50	No. 100	No. 180	Light	Heavy	24 hours	7 days	28 days	3 months	6 months	12 months
Portland A.......	13											
	14											
	15	0.15	5.4	21.2	12	207	371	655	875	941	720	787
	16	..	..	..	29	297	303	750	973	1008	735	816
	18	..	..	..	80	355	260	649	773	831	645	748
	20	..	..	..	142	402	233	500	693	716	621	676
	22	..	..	..	268	473	184	546	635	658	601	589
	24	..	..	..	327	912	167	539	649	644	629	755
Portland B.......	13	0.1	7.0	18.0	13	270	366	775	859	1067	892	832
	14	..	..	..	18	303	404	780	891	972	852	781
	15											
	16	..	..	..	22	327	363	602	725	844	806	723
	18	..	..	..	15	383	308	570	723	785	728	724
	20	..	..	..	56	703	225	590	718	760	674	636
	22	..	..	..	52	833	166	554	649	731	643	604
	24	..	..	..	188	918	42	510	691	695	632	574
Natural (Lehigh Valley)	23	0.1	4.6	10.2	13	32	212	251	252	311	275	356
	24											
	25	..	..	..	18	39	185	218	215	289	300	341
	27	..	..	...	21	42	150	188	220	257	272	314
	29	..	..	..	20	52	128	178	202	246	248	256
	31	..	..	..	21	57	112	173	199	224	259	309
	33	..	..	..	27	85	104	172	182	267	246	290
	35	..	..	..	38	137	93	121	178	260	286	319
	37	..	..	..	34	160	85	108	168	262	306	326
	39	..	..	..	67	233	85	119	202	252	371	400
Natural (Rosendale)	23	2.3	12.4	21.9	22	59	138	177	271	332	284	264
	24	..	..	..		78	125	141	264	342	309	310
	25	..	..	..	35	120	150	164	216	308	318	321
	27	..	..	..	49	143	117	116	194	305	345	272
	29	..	..	..	76	166	96	105	164	272	320	267
	31	..	..	..	117	212	72	72	159	270	371	225
	33	..	..	..	115	235	62	71	147	277	379	244
	35	..	..	..	127	400	50	64	112	245	318	315
	37	..	..	..	198	828	59	62	96	..	284	351
	39	..	..	..	260	1057	54	56	85	..	355	364

Note. — Results shown are the averages of six briquettes made.

In some cases the form of the grain[*] and the mineralogic composition[†] may exert a certain influence, although tests show that these are usually of inferior importance to the mechanical or granulometric composition of the sand or screenings. It is possible that the fine dust or impalpable powder in certain stone may chemically react upon the cement.

The effect of mica in screenings from broken stone containing this material has been the subject of considerable controversy. Experiments by Mr. Feret[*] in France indicate that the presence of 2% of mica has but slight, if any, influence upon the tensile strength of the mortar, but a greater one upon its compressive strength.

SHARPNESS OF SAND

In the past all specifications have called for clean, "sharp" sand in spite of the fact that in many parts of the country where sharp sand is not obtainable, sand with rounded grains is furnished and used with perfect satisfaction.

Comparative laboratory tests under conditions as nearly as possible identical uphold the practice of using sand with rounded grains. They indicate, as may be inferred from the previous discussion in this chapter, that the chief difference in natural sands is due to the size of the grains, and while the sharpness of grain may exert a certain influence it is of so much less importance than the size of the grain that *the requirement of sharpness for sand should be omitted from concrete specifications.*

Referring to columns (11) and (22) in the table on page 136, and to Fig. 49, page 140, it is evident that the difference in strength of nearly all the mortars made with the various sands is explained by the differing percentages of cement and densities without reference to the character of the grains. The only noticeable exception is with the artificial sand, M′, which consists of mixed sizes of crushed quartz. Mr. Feret[‡] believes that this exception may be due to chemical action produced by the large quantity (⅙ its weight) of impalpable quartz. Sand N′, also crushed quartz, but containing none of this fine powder, produces a mortar similar in strength to like mortars of natural sand having rounded grains.

Other tests of Mr. Feret[§] and comparative tests, in the United States, of

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[*] Baumaterialienkunde, V Jahrgang (1900), p. 21, and Annales des Ponts et Chaussées, 1892, II, p. 124.

[†] Mr. P. Alexandre found calcareous sands to give relatively high strength, and Mr. Feret obtained similar high results with marble.

[‡] Bulletin de la Société d'Encouragement pour l'Industrie Nationale, 1897, Vol. II.

[§] Annales des Ponts et Chaussées, 1892, II, p. 124.

mortar with crushed quartz and natural sands generally confirm the above conclusion. The variation in the shape of the grains of natural sands and crushed quartz is illustrated in Figs. 62, 64, and 65, page 175.

EFFECT OF NATURAL IMPURITIES IN THE SAND UPON THE STRENGTH OF MORTAR

A clause to the effect that a sand for mortar or concrete shall be "clean" is almost universally found in masonry specifications. The necessity for this requirement is often questioned by cement experimenters, because the results of tests of mortar to which percentages of loam or clay have been added, often give higher results than those of mortar made with cement and pure sand.

As a matter of fact, it is impossible to make a general statement either to the effect that loam or clay is beneficial or that it is detrimental to cement mortars. In some cases it is undoubtedly an actual benefit, while in others the contrary is true, chiefly depending upon the richness of the mortar and the coarseness of the sand. Lean mortars may be improved by small admixtures of loam or clay or by substituting dirty for clean sand, because the fine material increases the density. Rich mortars, on the other hand, do not require the addition of fine material, and it may be positively detrimental, because the cement furnishes all the fine material required for maximum density. This is illustrated in experiments by Mr. Griesenauer* in which an admixture of even 2% of loam (based on the weight of the sand) slightly reduced the strength of 1: 2 mortar, while 20% of loam, added to the 2 parts of sand, reduced the strength about 30%. In 1: 3 mortar, on the other hand, the addition of 2% slightly increased the strength, and there was no appreciable injury up to 20% addition.

In experiments by Mr. E. S. Wheeler† clay reduced the strength of neat and 1: 1 mortars, but improved leaner mixtures.

EFFECT OF LIME UPON THE STRENGTH OF MORTAR

As a principal constituent of mortar in masonry construction, lime is inferior to cement in durability and strength. However, not only because of its relative cheapness, but also because a small addition of slaked or hydrated lime may increase the density of the mortar and cause it to work easier under the trowel, a limited quantity often can be used to advantage in mortar which is to be subjected to high loading.

***Engineering News*, April 28, 1904, p. 413.

†Report Chief of Engineers, U. S. A., 1895, p. 3004, and 1896, p. 2827.

For concrete, lime has been suggested, as mentioned in Chapter XX, on Water-tightness, as a suitable ingredient to fill the voids and thus render it more impermeable.

Although lime mixed with neat cement is apt to decrease its strength, in combination with sand for cement mortars, a small admixture of lime may add to the strength of the mortar. The questions as to whether lime is beneficial, and as to the amount which can be used, are determined by the character of the cement, the coarseness of the sand, and the proportions in which the two are mixed. The effect of lime in cement mortar or concrete is chiefly mechanical. In a porous mortar or concrete a small quantity of it assists in filling the voids, and if it is thoroughly slaked so as to contain no quicklime, its expansion need not be feared.

Since even a neat cement paste has 35% to 45% water plus air voids, the inference might be drawn that the addition of lime would increase its density, and thus that the lime would be valuable even in very rich mortars. However, it seems to be practically impossible, except under high pressure, to replace the water which occupies the voids in neat cement paste with lime or any other fine powder. But it is evident that a lean mortar, such as a 1:4, or even a 1:3, should be improved by the addition of lime, and that this is true is illustrated in the following tests by Mr. E. S. Wheeler.* In these experiments the addition of 10% of lime — based on the weight of the cement — increases the strength of 1:3 mortar, and as shown by item (3) in the table, a 1:3⅓ mortar with 10% of lime is stronger than a 1:3 mortar with no lime. Items (4) and (5) illustrate the reduction in

Effect of Lime Paste upon the Strength of Portland Cement Mortar.
By E. S. Wheeler. (*See p.* 155.)

Item	Proportions cement plus lime to sand by weight	Proportions cement to sand by weight	Cement	Lime†	Sand	Average Tensile Strength.	
						at 28 dys.	at 3 mos.
	parts	parts	grams	grams	grams	lb. per sq. in.	lb. per sq. in.
(1)	1:3	1:3	200	0	600	201	236
(2)	1:2¾	1:3	200	20	600	242	265
(3)	1:3	1:3⅓	180	20	600	238	264
(4)	1:3	1:4	150	50	600	168	171
(5)	1:3	1:6	100	100	600	57	70

*Report Chief of Engineers, U. S. A., 1896, p. 2823.

†The weight of the lime paste was 2.7 times the weights in this column.

strength when the lime becomes more nearly a principal ingredient. Each value is an average of five briquettes.

With another brand of cement and sand of different coarseness the relative quantity of lime to produce similar results will differ, but the general principle will still hold. In determining the amount of lime to add without decreasing the strength of a certain mortar, tests should be made with the materials to be employed.

In scientific experiments by Mr. Feret* the maximum strength of 1:4 mortar of Portland cement and sand from Saint Malo† was reached with an addition of 4% or 5% by weight of hydrated lime powder. As the mortar became richer, the lime had less effect, until at proportions 1:2, the addition of lime reduced the density, and at proportions 1:1½ the strength was also lowered.

A larger number of bricks can be laid in a given time with mortar containing lime than with a lean cement mortar because the lime fills the pores in the mortar so that it spreads more readily without crumbling and adheres better to the bricks in " buttering " them.

Unslaked Lime. Unslaked lime mixed with cement either for mortar or concrete is liable to produce expansion in the masonry. Builders recognize that lime, putty, or paste is much improved by standing for several days, or, better, for months, before being used, because all the small lumps are thus slaked. This thorough slaking is especially necessary when lime is to be used, even as a very small ingredient, in important concrete and masonry construction; an admixture of even 2% of ground quicklime may seriously reduce the strength of the mortar.‡

Weight and Volume of Lime. In proportioning lime to cement, the method of measurement must be clearly stated. The volume of common lime or quicklime increases in slaking to about 2½ times its volume measured loose in the lime cask, the exact increase varying with the chemical composition and the purity of the lime. The weight of lime paste is about 2½ times the weight of the same lime before slaking. Hydrated lime powder also occupies more volume than quicklime from which it is made.

GROUND TERRA-COTTA OR BRICK AS A SUBSTITUTE FOR SAND

Experiments by Mr. E. S. Wheeler§ indicated that for a mortar of light weight terra-cotta may be ground and used instead of sand. Tests with

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*Chimie Appliquée, 1897, p. 481.

†See p. 137.

‡Report Chief of Engineers, U. S. A., 1895, p. 2999.

§Report Chief of Engineers, U. S. A., 1896, p. 2866.

both Portland and Natural cement mixed with the ground terra-cotta in various proportions gave at the end of three months tensile strengths which are not appreciably different from the strengths obtained with standard crushed quartz. Red brick pulverized* may also be used for the same purpose with good results.

EFFECT OF REGAGING MORTAR AND CONCRETE

Engineers have frequently specified and insisted that concrete or mortar be used immediately, that is, within one hour or one-half hour after it is gaged. As opposed to this requirement, tests by various experimenters indicate with singular unanimity that, at least for Portland cements, it is unnecessary, and that Portland cement concrete or mortar may remain for at least two hours in the mortar bed without deterioration. In fact, the ultimate tensile and compressive strength appears to be thus increased. The results of such tests lead to the following conclusions:

(1) The tensile or compressive strength of Portland cement mortars or concretes is not lowered by standing two hours after mixing.
(2) Continuous gaging increases the ultimate strength.
(3) Regaging makes the cement slower setting.

With Natural cements, however, the results of experiments are somewhat contradictory. It is probable that some Natural cements are injured, and, therefore, if circumstances require delay in placing Natural cement mortar, the effect of such delay should be determined by tests upon the brand to be used.

Mr. E. Candlot (see page 124) states that the adhesive quality of cement mortar is reduced by regaging.

Extended tests to determine the effect of regaging neat cements and mortars have been made by Mr. P. Alexandre† and Mr. E. Candlot‡ in France, by Mr. Henry Faija§ in England, by Mr. James E. Howard¶ at the Watertown Arsenal, U. S. A., and by Mr. Thomas F. Richardson at the Wachusett Dam, Massachusetts.

Mr. Richardson in the course of his experiments made a batch of 1:2 mortar from each cement, cut it into two portions and, leaving half of it in

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*Report Chief of Engineers, U, S. A., 1896, p. 2830.

†Annales des Ponts et Chaussées, 1890, II, p. 340.

‡Candlot's Ciments et Chaux Hydrauliques, 1898, p. 355.

§Butler's Portland Cement, 1899, p. 307.

¶Tests of Metals, U. S. A., 1901, p. 497.

the mortar box, had the other half worked continuously. At various periods ranging from seven minutes to two hours, samples were taken from each portion, and made into tensile briquettes. Several brands of American and English Portland cements, both slow and quick-setting, and several brands of Natural cement having different periods of set, were tested. Referring to the results Mr. Richardson states:*

> For the quicker setting cements there was a considerable falling off in strength in the briquettes broken seven days after being mixed, and a somewhat less falling off for those broken twenty-eight days after mixing; but at the age of six months all the mortars which had been allowed to stand, or which were worked continuously for one and one-half and two hours, showed a considerable gain in tensile strength.

A typical series of tests with Rosendale cement, which attained its initial set in forty minutes and its final set in ninety minutes, and coarse sand (passing a No. 8 and retained on a No. 30 sieve) is presented in the following table:

Effect of Regaging upon the Tensile Strength of 1 : 2 *Natural* (*Rosendale*) *Cement Mortar.* (*See p.* 158.)

By Thomas F. Richardson.

Age	Periods of Sampling.				
	Immediately	After one hour		After two hours	
		Worked	Not Worked	Worked	Not Worked
	lb. per sq. in.	lb. per sq. in.	lb. per sq. in.	lb. per sq. in.	lb. per sq. in.
7 days..........	27	23	21	19	15
28 days..........	22	34	27	32	29
3 months........	120	155	141	192	150
6 months........	163	223	191	225	213

As a result of his tests, Mr. Richardson allowed the contractor, when necessary, to use the mortar on the dam up to two hours after being mixed. This was often a great convenience because of the distance of the mortar-mixing machine from the dam.

Mr. Howard at the Watertown Arsenal took samples of neat Portland

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*Personal correspondence.

cement after longer periods of setting, in some cases up to one hundred and two hours. In general, his specimens showed at the age of one month no appreciable difference, whether they were taken when first gaged or at four, or in some cases eight, hours after gaging. The strength of specimens taken after longer periods of standing was found at the age of one month to be lower. Natural cements showed an immediate falling off, due to regaging, on the thirty days' tests, but the tests were not extended beyond this age.

The Setting of Regaged Mortars. The experiments of Mr. Candlot were made chiefly upon mortars which had attained their final set, as determined by the pressure of the thumb. These mortars, after regaging, set much more slowly than normally gaged mortars, and he states that the set occurred at approximately the same time with all cements. "Thus, whether a mortar originally sets in ten minutes or three hours, when regaged it requires, in either case, about eight to ten hours." He concludes from this action that, in Portland cements, aluminate of lime, which plays an important part in the setting, has no action on the hardening.

> Consequently regaging should have little influence upon siliceous products, while it would be expected to seriously affect aluminous cements. This is the effect in practice, for limes and Portland cements can be regaged without bad results, while the strength of Natural Vassy cement is considerably lowered by regaging.*

Effect of Regaging upon Adhesion. Mr. Candlot* found that mortars which had set several hours before molding, although usually showing as great compressive or tensile strength as normal mortars, gave much lower strength in adhesion, the reduction in strength being often 50%. (See p. 124.)

EFFECT OF GAGING WITH SEA WATER

Mr. Alexandre† concludes from his own and other experiments which extend to a three-year period, that there is no essential difference in strength of mortars gaged with fresh and with sea water. Briquettes gaged with sea water, however, usually set very much slower than those gaged with fresh water.‡

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*Candlot's Ciments et Chaux Hydrauliques, 1898, pp. 358 and 360.

†Annales des Ponts et Chaussées, 1890, II, p. 332.

‡Alexandre and Feret in Commission des Méthodes d'Essai des Matériaux de Construction, 1895, Vol IV, p. 111.

CHAPTER X

VOIDS AND OTHER CHARACTERISTICS OF CONCRETE AGGREGATES

In this chapter are given tables of the specific gravities and voids of different materials, and the method of determining them, also laws relating to the voids in concrete aggregates, and the effect of compacting such materials.

Laws of Volumes and Voids. The most important of these general laws relating to volumes of different materials, and to their voids, may be stated as follows:

(1) A mass of equal spheres, if symmetrically piled in the theoretically most compact manner, would have 26% voids whatever the size of the spheres, but by experiment it is found that it is practically impossible to get below 44% voids. (See p. 168.)

(2) If a dry material having grains of uniform shape be separated by screens into grains of uniform dimensions, the separated sizes (except when finer than will pass a No. 74 screen) will contain approximately equal percentages of voids; in other words, a dry substance consisting of large particles, all of similar size and shape, will contain practically the same percentage of voids as a substance having grains of the same shape but of uniformly smaller size. (See p. 170.)

(3) In any material the largest percentage of voids occurs with grains of uniform size, and the smallest percentage of voids with a mixture of sizes so graded that the voids of each size are filled with the largest particles that will enter them. (See p. 171.)

(4) An aggregate consisting of a mixture of coarse stones and sand has greater density — that is, contains a smaller percentage of voids — than the sand alone. (See p. 172.)

(5) By Fuller's experiments, perfect gradation of sizes of the aggregate appears to occur when the percentages of the mixed aggregate passing different sizes of sieves are defined by a curve which approaches a parabola. (See Chap. XI, p. 195.)

(6) Materials with round grains, such as gravel, contain fewer voids than materials with angular grains, such as broken stone, even though

the particles in both may have passed through and been caught by the same screens. (See p. 174.)

(7) The mixture of a small amount of water with dry sand increases its bulk. In the case of most bank sands the maximum volume — and hence the smallest amount of solid matter per unit of volume, that is, the largest percentage of absolute voids — being reached with from 5% to 8% of water. (See p. 176.)

CLASSIFICATION OF BROKEN STONE.*

Rocks which are commonly employed for concrete or for road making are commercially classified as (*a*) traps, (*b*) granites, (*c*) limestones, (*d*) conglomerates, and (*e*) sandstones.

The trade term "trap" includes dark green to black, heavy, close textured, tough rocks of igneous origin, thus covering a variety of rock whose mineralogical names are diabase, norite, gabbro, etc. As shown in the table below, the traps usually range in specific gravity from 2.80 to 3.05.

Granites, commercially so called, include the lighter colored, less dense rock, such as not only true granite, but syenite, diorite, gneiss, mica schist, and several other groups. Their specific gravities range from about 2.65 to 2.85, averaging close to 2.70. Although, as road metal, the traps are usually far superior to granites, for concrete there appears to be no great difference in the value of the two classes. The distinction, however, is worth keeping because a concrete stone is often purchased from road metal quarries.

Limestones of normal type range in specific gravity from 2.47 to 2.76, averaging about 2.60, although the very soft stones, which are not suitable for high class concrete, may fall below 2.0.

Conglomerate, or pudding stone as it is often termed, is essentially a very coarse grained sandstone, ranging in specific gravity from 2.50 to 2.80. It makes a good concrete aggregate.

Sandstones of compact texture, such as the Potsdam and Medina sandstones, and the Hudson River bluestone, may run as high in specific gravity as 2.75, while the looser textured, more porous sandstones may fall as low as 2.10, a fair average being about 2.40.

Shale and slate make poor concrete aggregates, because their crushing and shearing strength is low.

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*The authors are indebted to Mr. Edwin C. Eckel for the material under this heading, which has been especially prepared by him for this Treatise.

Specific Gravity of Stone from Different Localities.

Compiled by Edwin C. Eckel.

TRAP.

Locality.	Specific Gravity.
Massachusetts	
Boston	2.78
Minnesota	
Duluth	3.00
Duluth	2.80
Taylors Falls	3.00
New Jersey	
Jersey City Heights	3.03
Little Falls	2.99
New York	
Staten Island	2.86

GRANITE.

Locality.	Specific Gravity.
California	
Penrhyn	2.77
Rocklin	2.68
Connecticut	
Greenwich	2.84
New London	2.66
Georgia	
Stone Mt.	2.69
Maine	
Hallowell	2.66
Maryland	
Port Deposit	2.72
Massachusetts	
Quincy	2.70
New Hampshire	
Keene	2.66
New York	
Ausable Forks	2.76
Rhode Island	
Westerly	2.67
Vermont	
Barre	2.65
Wisconsin	
Amberg	2.71
Montello	2.64

LIMESTONE.

Locality.	Specific Gravity.
Illinois	
Joliet	2.56
Lemont	2.51
Quincy	2.57
Indiana	
Bedford	2.48
Salem	2.51
Minnesota	
Frontenac	2.63
Winona	2.67
New York	
Canajoharie	2.68
Glens Falls	2.70
Kingston	2.69
Prospect	2.72
Sandy Hill	2.76
Williamsville	2.71
Soft Limestone	
France	
Caen	1.84

SANDSTONE.

Locality.	Specific Gravity.
Colorado	
Ft. Collins	2.43
Trinidad	2.34
Connecticut	
Portland[1]	2.64
Massachusetts	
Longmeadow[1]	2.48
Minnesota	
Fond du Lac	2.24
New Jersey	
Belleville[1]	2.26
New York	
Albion[2]	2.60
Medina[2]	2.41
Potsdam[3]	2.60
Oxford[4]	2.71
Malden[5]	2.75
Oswego	2.42
Ohio	
Berea[6]	2.14
Cleveland	2.21
Massillon	2.11

[1] Brownstone.

[2] Medina sandstone.

[3] Potsdam sandstone.

[4] Bluestone.

[5] Hudson River Bluestone.

[6] Berea grit.

AVERAGE SPECIFIC GRAVITY OF SAND AND STONE

The specific gravity of a substance is the ratio of the weight of a given volume to the weight of the same volume of distilled water at a temperature of 4° Cent. (39° Fahr.). For ordinary tests of stone and sand, the water need not be distilled and may be at ordinary temperature.

A knowledge of the specific gravity of the particles of the sand and stone is important to the engineer as a ready means of determining the percentages of voids.

The uniformity in the specific gravity of different sands is very con venient for calculation. Different authorities who have tested large quantities of sand have reached almost identical conclusions as to the average specific gravity, and all state that it is practically a constant. Mr. Allen Hazen gives 2.65, Mr. William B. Fuller, 2.64, Mr. R. Feret in France states that "one may without appreciable error adopt an average specific gravity of 2.65 for siliceous sands,"* while Mr. E. Candlot gives limits of 2.60 to 2.68 for sands which are not porous.† The specific gravity of calcareous sands averages about 2.69 by absolute determination, or about 2.55 if measured by the total volume of the particles having their pores filled with air.

Gravels also have quite uniform specific gravity. According to Mr. A. E. Schutté, who has tested gravel from more than forty localities in the United States and Canada, an average value is 2.66.

The following table gives average values of various concrete aggregates. In every case, the specific gravity is the ratio of the weight of an absolutely solid unit volume of each material to the weight of a unit volume of water. Specific gravities of stone from various localities are given on page 162.

Average Specific Gravity of Various Aggregates. (See p. 163.)

Material.	Specific Gravity.	Weight of a solid cu. ft. of rock. lb.	Authority.
Sand	2.65	165	Allen Hazen
Gravel	2.66	165	A. E. Schutté
Conglomerate	2.6	162	Robert Spurr Weston
Granite	2.7	168	Edwin C. Eckel
Limestone	2.6	162	Edwin C. Eckel
Trap	2.9	180	Edwin C. Eckel
Slate	2.7	168	Tod's Tables‡
Sandstone	2.4	150	Edwin C. Eckel
Cinders (bituminous)	1.5	95	The authors

*Bulletin de la Société d'Encouragement pour l'Industrie Nationale, 1897, Vol. II, p. 1591.

†Ciments et Chaux Hydrauliques, 1898, p. 246.

‡Encyclopedia Britannica.

METHOD OF DETERMINING SPECIFIC GRAVITY

The specific gravity of a sample of material is determined by dividing its weight by the weight of water which it displaces when immersed.

The size of sample necessary for the accurate determination of a sand or stone of fairly uniform texture depends chiefly upon the delicacy of the apparatus employed. If scales reading to grams, and measures reading to cubic centimeters, are employed, a sample of 250 grams should give accurate results to two decimal places. With scales reading to ¼ ounce, a sample of 4 lb. is necessary for similar accuracy. The water must be maintained at 68° Fahr. (20° Cent.).

The sample should be taken by the method of quartering described on page 280.

Before finding the specific gravity of siliceous sand, the sample should be dried in an oven at a temperature as high as 212° Fahr. (100° Cent.) until there is no further loss in weight. A porous stone, on the other hand, may be first moistened sufficiently to fill its pores, and then the surfaces of the particles dried by means of blotting paper. If this method is followed, the material should be in a similar condition when its voids are determined by the method given on page 165. The absolute specific gravity of the porous stone may be afterward found by drying in an oven and correcting for the moisture lost.

The apparent specific gravity of sand or stone may be determined with an apparatus consisting of scales reading to ¼ ounce or to 5 grams, and a tall glass vessel with a reference mark, such as a cylinder or a pharmacist's graduate. The method is as follows:

Make a mark at any convenient place on the neck of the vessel;
Fill the vessel with water at a temperature of 68° Fahr. (20° Cent.) up to this mark;
Take a known weight in grams or ounces of the material;
Pour material into vessel carefully, a few grains at a time, so that no bubbles of air are carried in with it;
Pour out the clear water displaced by the material (leaving water in the vessel up to the level of the mark), and weigh the water poured out.

Let

S = Weight of material placed in vessel.
W = Weight of water displaced.

Then

$$\text{Specific gravity of material} = \frac{S}{W} \qquad (1)$$

It is essential that the weight of water displaced be weighed to within ±2%. If the scales are not sufficiently sensitive, more material must be taken and a larger vessel used. With balances sensitive to 1 gr. or 1/16 oz. the displacement of more than 3 ounces of water is necessary.

METHOD OF DETERMINING VOIDS

The voids in sand, gravel, and broken stone may be obtained directly from the tables on pages 166 and 167. Special determinations may be made as described below.

The percentage of voids in sand or fine broken stone cannot be accurately obtained by the ordinary method of placing in a measure and pouring in water, because it is physically impossible to drive out all the air. There may be enough of this held to amount to 10% of the volume of the sand, and thus cause a corresponding error in the percentage of voids.

The voids in coarse stone containing no particles under ½-inch diameter may be determined by placing in a box or pail of known volume and pouring in water, but if the specific gravity is known, the method described below is simpler and more accurate.

The only apparatus required are scales of fair accuracy and an exact measure which contains not less than ½ cu. ft. If a cubic foot measure is not available a 16-quart pail will answer the purpose, although compactness of the sand is less easily adjusted because of the small diameter. Such a pail holds slightly over ½ cu. ft. and the exact measure is determined by weighing the pail, pouring in 31 lb. 2 oz. of water, and marking the level of the surface. The pail up to this mark contains ½ cu. ft. of any material.

The method of determining the voids is as follows:

Weigh the measure;

Fill the measure to the required level with the material in the state in which the percentage of voids is required, that is, loose, shaken, or packed;

Weigh, and deduct the weight of the measure, calling the net weight of a cubic foot of the material, S;

If the material consists of, or contains, sand or fine stone, correct for moisture by taking an exact weight, — about 10 lb., — drying in an oven at a temperature of at least 212° Fahr. (100° Cent.) until there is no further loss in weight, and after calculating the percentage of moisture in terms of the weight of the original moist sand or stone, express the percentage as a decimal, p.

Select the weight of a cubic foot of absolutely solid rock* from the table on page 163, and call it R.

$$\text{Per cent of absolute voids} = \left(1 - \frac{S - Sp}{R}\right)100 \tag{3}$$

The air voids are determined, if desired, by deducting the volume of moisture (its weight divided by the weight of one cubic foot of water)

Percentages of Voids Corresponding to Different Weights per Cubic Foot of Sand, Gravel, and Broken Stone Containing Various Percentages of Moisture. (*See p.* 168.)

Weight of one cu. ft. of sand or gravel.†	PERCENTAGES OF ABSOLUTE VOIDS IN MATERIAL CONTAINING MOISTURES BY WEIGHT.‡					Moisture by volume corresponding to 1% by weight.‡
	0%	2%	4%	6%	8%	
	%	%	%	%	%	%
70	57.6	58.4	59.3	60.1	61.0	1.1
75	54.5	55.4	56.4	57.3	58.2	1.2
80	51.5	52.5	53.4	54.4	55.4	1.3
81	50.9	51.9	52.9	53.9	54.8	1.3
82	50.3	51.3	52.3	53.3	54.3	1.3
83	49.7	50.7	51.7	52.7	53.7	1.3
84	49.1	50.1	51.1	52.2	53.2	1.4
85	48.5	49.5	50.6	51.6	52.6	1.4
86	47.9	48.9	50.0	51.0	52.0	1.4
87	47.3	48.3	49.4	50.4	51.5	1.4
88	46.7	47.7	48.8	49.9	50.9	1.4
89	46.1	47.1	48.2	49.3	50.4	1.4
90	45.5	46.5	47.6	48.7	49.8	1.4
91	44.8	45.9	47.0	48.2	49.2	1.5
92	44.2	45.4	46.5	47.6	48.7	1.5
93	43.6	44.8	45.9	47.0	48.1	1.5
94	43.0	44.2	45.3	46.5	47.6	1.5
95	42.4	43.6	44.7	45.9	47.0	1.5
96	41.8	43.0	44.1	45.3	46.4	1.5
97	41.2	42.4	43.6	44.7	45.9	1.6
98	40.6	41.8	43.0	44.2	45.3	1.6
99	40.0	41.2	42.4	43.6	44.8	1.6
100	39.4	40.6	41.8	43.0	44.2	1.6
101	38.8	40.0	41.2	42.5	43.7	1.6
102	38.2	39.4	40.7	41.9	43.1	1.6
103	37.6	38.8	40.1	41.3	42.5	1.6
104	37.0	38.2	39.5	40.8	42.0	1.7
105	36.4	37.6	38.9	40.2	41.4	1.7
106	35.8	37.0	38.3	39.6	40.9	1.7
107	35.2	36.4	37.7	39.0	40.3	1.7
108	34.6	35.9	37.2	38.5	39.7	1.7
109	33.9	35.3	36.6	37.9	39.2	1.7
110	33.3	34.7	36.0	37.3	38.7	1.8
115	30.3	31.7	33.1	34.5	35.9	1.8
120	27.3	28.7	30.2	31.6	33.1	1.9
125	24.2	25.8	27.3	28.8	30.3	2.0
130	21.2	22.8	24.4	25.9	27.5	2.1
135	18.2	19.8	21.4	23.1	24.7	2.2
140	15.2	16.8	18.5	20.2	21.9	2.2

---

*The weight per cubic foot of a solid is the specific gravity of the rock multiplied by the weight of a cubic foot of water.

†Also applicable to broken stones such as granite, conglomerate, and limestone, whose specific gravity averages from 2.6 to 2.7. Table is based on specific gravity of 2.65.

‡The per cent. of absolute voids given in the columns include the space occupied by both the air and the moisture. To determine the per cent. of air space, multiply the figure in the last column, opposite the weight of sand under consideration, by the per cent. of moisture by weight, and deduct result from the per cent. already found.

in a unit volume of the sand or stone, from the total voids. Expressed in percentages with notation same as above,

$$\text{Per cent. of air voids} = \text{Per cent. of absolute voids} - \frac{Sp}{62.3}\,100 \qquad (4)$$

Example. — Given a sand whose loose weight per cubic foot is found to be 92 lb. and its moisture 3% by weight. Find the percentage of voids in the loose sand.

Solution by formula. — Since from the example $S = 92$ and $p = 0.03$, and, from table on page 163, $R = 165$,

$$\text{Percentage of absolute voids} = \left(1 - \frac{92 - 0.03\,(92)}{165}\right)100$$

$$= 45.9\%$$

This percentage includes the space occupied by the moisture. The net percentage of voids occupied by air alone is the difference between the absolute voids and the percentage of moisture by volume. Moisture is $92 \times 0.03 = 2.76$ lb., or $\frac{2.76}{62.3} = 0.044$ cu. ft., corresponding to 4.4% voids by volume, hence air voids are 45.9% — 4.4% = 41.5%.

Percentages of Voids Corresponding to Different Weights per Cubic Foot of Dry Broken Stone of Various Specific Gravities. (*See p.* 168.)

Weight of one cu. ft. of dry broken stone.	PERCENTAGES OF ABSOLUTE VOIDS CORRESPONDING TO SPECIFIC GRAVITIES OF STONE OF					
	2.4*	2.5	2.6†	2.7‡	2.8	2.9§
	%	%	%	%	%	%
70	53.2	55.0	56.8	58.4	59.9	61.3
75	49.8	51.8	53.7	55.4	57.0	58.5
80	46.5	48.6	50.6	52.4	54.1	55.7
85	43.2	45.4	47.5	49.5	51.3	53.0
90	39.8	42.2	44.5	46.5	48.4	50.2
95	36.5	39.0	41.4	43.5	45.5	47.4
100	33.1	35.8	38.3	40.6	42.7	44.7
105	29.8	32.6	35.2	37.6	39.8	41.9
110	26.4	29.4	32.1	34.6	36.9	39.1
115	23.1	26.2	29.0	31.6	34.1	36.4
120	19.8	23.0	25.9	28.7	31.2	33.6
125	16.4	19.8	22.8	25.7	28.3	30.8
130	13.1	16.6	19.8	22.7	25.5	28.1
135	9.7	13.3	16.7	19.7	22.6	25.3
140	6.4	10.1	13.6	16.8	19.7	22.5

NOTE.—Average specific gravity of bituminous coal cinders may be taken as 1.5.

*Sandstone. ‡Granite and slates.

†Limestone and conglomerates. §Trap.

Solution by table (p. 166.) — Opposite 92 lb. per cu. ft., interpolating between 2% and 4% moisture, is 46.0% of absolute voids. From last column 3% by weight corresponds to 3% x 1.5 = 4.5% by volume. 46.0% — 4.5% = 41.5% air voids.

Tables of Voids. From the tables on pages 166 and 167, the voids in sand, gravel, and broken stone may thus be determined simply by weighing the material and finding the percentage of moisture contained in it, as above described. Since the percentage of moisture by volume is always greater than its percentage by weight, and the two are not proportional to each other, the final column is inserted in the first table for convenience in calculating the moisture by volume.

VOIDS AND DENSITY OF MIXTURES OF DIFFERENT SIZED MATERIALS

The term *density* as applied to mortar is defined on page 135. Similarly, in a dry material, such as a concrete aggregate, it is represented by the total volume of the solid particles entering into a unit volume of the aggregate. In dry materials the density is the complement of the voids, since a material which has, say, 40% voids will have a density of 0.60; but density is a more correct term to use than voids because it is applicable to concretes and mortars in which connection the term voids is somewhat ambiguous. The example on page 139 illustrates the method of determining the density of a concrete or mortar.

The densities of dry aggregates of uniform specific gravity, or of mixtures in uniform proportions of materials with different specific gravities, are in direct proportion to their weights. For example, the densities of different dry sands may be compared by weight; or the densities of different mixtures of sand and broken trap in proportions, say, 2 parts sand to 4 parts trap may be compared by weight; but the density of sand and the density of trap screenings cannot be compared by weights unless the differing specific gravities are taken into account.

In the following discussion of the laws formulated on page 160, both the terms *density* and *voids* are used in relation to the dry materials.

Voids in Masses of Similar Sized Particles. (1) The fact that the percentage of voids in a mass of equal spheres symmetrically piled in the theoretically most compact manner is independent of the actual diameter is simply a geometrical proposition, evident without demonstration by inspection of Fig. 58.

In actual experiment it has been found that while the percentage of voids is uniform regardless of the size of the spheres, it is impossible to

pour spheres into a measure so that they will arrange themselves symmetrically, and the rather astonishing result has been reached by Mr. Fuller (see p. 186) that 44% is the smallest percentage of voids which can be obtained with equal perfect spheres, no matter what may be their actual diameters or the size of the receptacle.

The following simple demonstration,* which is of theoretical interest, proves that the percentage of voids in a mass of equal spheres symmetrically piled in the most compact manner is 26%, and that the radii (and consequently the diameters) of the two next smaller spheres which can be inscribed between the larger ones are respectively 0.41 and 0.22 of the radius of the large spheres.

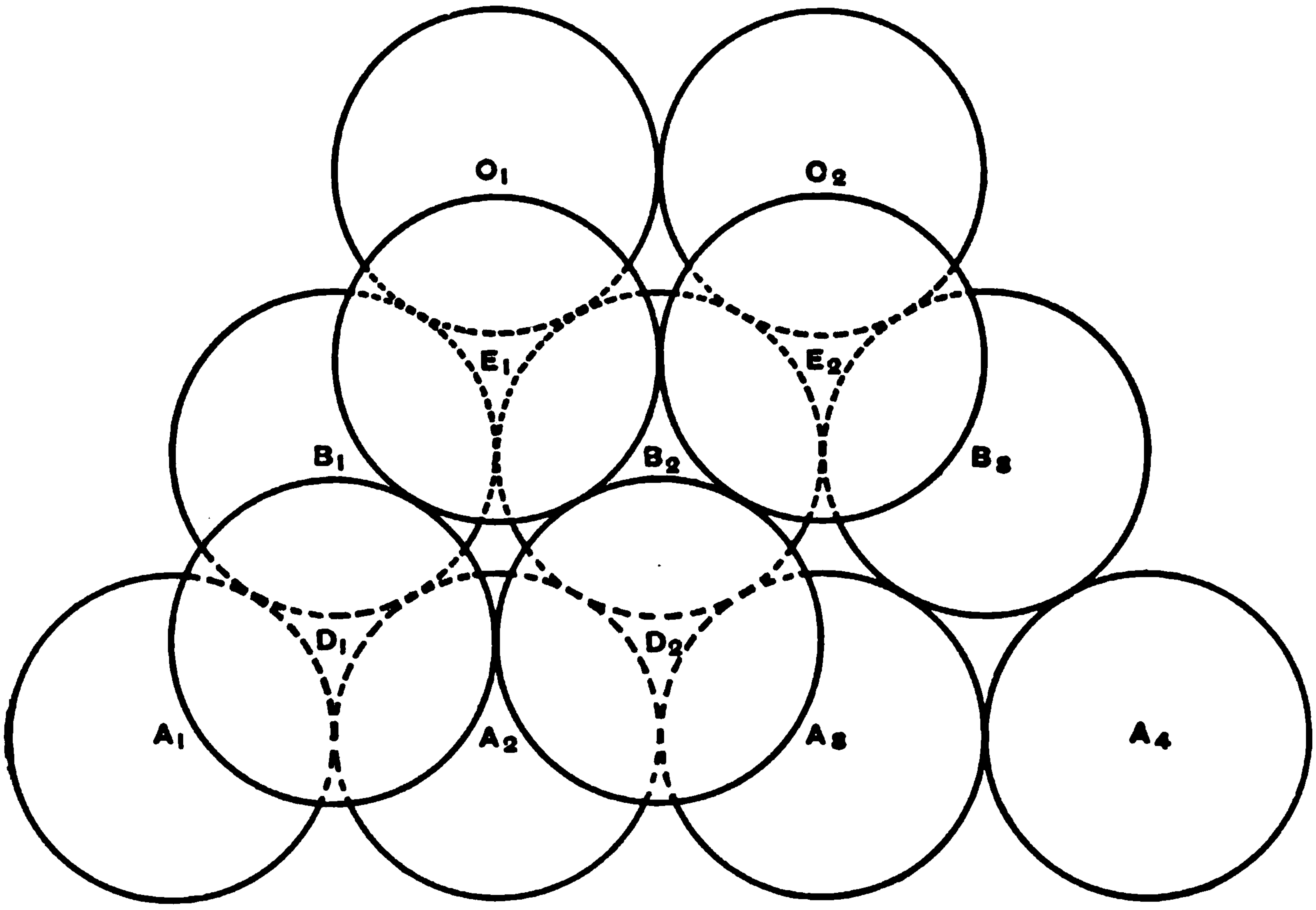

FIG. 58.—Spheres of Equal Size. (*See p.* 168.)

The circles in Fig. 58 represent a horizontal plan of two layers of spheres.

The centers A_1 A_2 B_1 D_1 form a regular tetrahedron.

Let edge be 2.

Altitude = difference between level of centers A, B, C, and level of centers D, E is $\frac{2}{3}\sqrt{6}$

Let number of spheres in a layer be *m*, number of layers *n*.

---

*For which the authors are indebted to Dr. Harry W. Tyler.

Volume of one sphere is $\frac{4\pi}{3}$

Volume of spheres in a layer, $\frac{4m\pi}{3}$

Volume of all spheres, $\frac{4mn\pi}{3}$ (approx.) $= V_1$

Cross-section of including space is $2\sqrt{3}\,n$ (approx.)

Volume of including space is $2\sqrt{3}\,n \times \frac{2}{3}\sqrt{6}\,m$ (approx.)

$= 4\sqrt{2}\,mn$ (approx.) $= V_2$

Ratio $\frac{V_1}{V_2} = \frac{4mn\pi}{3\times 4mn\sqrt{2}} = \frac{\pi}{3\sqrt{2}} = 0.74$ (approx.) corresponding to about 26% voids.

Inscribed Spheres.

1. Sphere inscribed between spheres A_1 A_2 B_1 and D_1:

Distance from any vertex A_1 of tetrahedron to center is $\frac{1}{2}\sqrt{6}$

Radius of small sphere $= \frac{1}{2}\sqrt{6} - 1 = 0.22$ (approx.) or about $\frac{22}{100}$ of the radius of the large spheres.

2. Sphere inscribed between A_2 B_1 B_2 and D_1 D_2 E_1:

Distance from A_2 to E_1 is $2\sqrt{2}$

Radius of small sphere $= \sqrt{2} - 1 = 0.41$ (approx.) or about $\frac{41}{100}$ of the radius of the large spheres.

(2) The proposition that if a dry material such as sand, pebbles, or irregular broken stone, having grains of fairly uniform shapes, be separated by screens into grains of uniform dimensions, the separated sizes will contain approximately equal percentages of voids, is not so self-evident, but experiment proves that in portions of the same material screened to uniform sizes the percentages of voids will be substantially alike until very fine sizes are reached, such as will pass a No. 74 sieve; below this degree of fineness the particles are entangled by air. The authors have found by experiments given in the following table, that different lots of broken stone from the same quarry, each screened to uniform size, will contain substantially the same percentages of voids, but that lots of stone from different quarries screened to the same size may differ because of the structure of the rock. Published records usually show slight variations in the weight per cubic foot of different sized broken stone, but it is noticeable that some authorities give the heaviest weight,

which corresponds to the smallest percentage of voids, for the larger sizes, while others give the reverse. For example, Patton's Civil Engineering gives the smallest percentage of voids in the coarsest broken stone, while Butler's Portland Cement gives the smallest percentage in the finest stone. The variation in results is undoubtedly due to differences in methods of compacting and to the variations in the sizes of the stones of each lot.

Experiments by Mr. Feret in France, and Mr. Thomas F. Richardson in the United States, show that the percentages of voids in absolutely dry sand which has been screened to uniform size are almost identical. Mr. Feret, experimenting by shoveling dry sand loosely into a 50 liter (1.8 cu. ft.) box, — a measure large enough to eliminate errors of placing, — found that fine (F) medium (M) and coarse (G) sands each contained about 50% voids, while mixing the sizes, which are defined on page 142, in the best proportions reduced the voids to 34%.

Voids and Compression of Broken Trap and Gravel. (*See p.* 170.)

Size of Stone	Class of Stone	Crusher	Size of Particles	Voids in loose stone %	Compression by light ramming or shaking %	Voids in lightly rammed or shaken stone %	Compression by heavy ramming %	Voids in heavily rammed stone %	
No. 2	HardTrap	Rotary	2½" to 1"	54.5			19.2	43.7	
No. 3		"	1" to ½"	54.5	14.3	46.9	20.5	42.8	
Nos. 2, 3, 4		"	2½" to dust*	45.0	14.5	35.7	20.8	30.6	
No. 2	Soft Trap	Jaw	2" to ¾"	51.2	11.9	44.6	17.8	40.6	Variation is due to trap breaking under rammer.
No. 3		"	¾" to ⅜"	51.2	14.3	43.1	23.9	35.0	
	Gravel		2½" to ½"	36.5	12.5†	27.4	11.5†	28.2	

Loose stone is as thrown by a laborer into a measuring box or barrel.
Material rammed in 6-inch layers.

Densest Mixture of Sand and Stone. (3) The fact that the densest mixture occurs with particles of different sizes is so evident as to require no proof, and this being recognized, it follows that the least density and hence the largest percentage of voids occurs when the grains are all of the same size. The converse of this proposition, that the smallest percentage of voids occurs in a mixture graded so that the voids of each size are filled with the largest particles which will enter them, is

*Mixed in proportions 44.4% No. 2, 33.3% No. 3, and 22.2% No. 4 (dust).

†Another gravel tested, compressed, 8.5% on shaking, and 11.2% on hard ramming.

illustrated in Figs. 59, 60, and 61, and is important in its application to the selection of materials for concrete.

(4) The fact that an aggregate consisting of a mixture of stones and sand has greater density, that is, contains fewer voids than the sand alone, is illustrated by comparison of Figs. 59 and 61. The voids of the large stone in Fig. 59 are filled with sand, while the voids in the same large stone in Fig. 61 are filled with mixed sand and stone, and the mass of the mixture is evidently denser, that is, it contains more solid material. This

FIG. 59.—Large Stones with Voids filled with Sand. (*See p.* 172.)

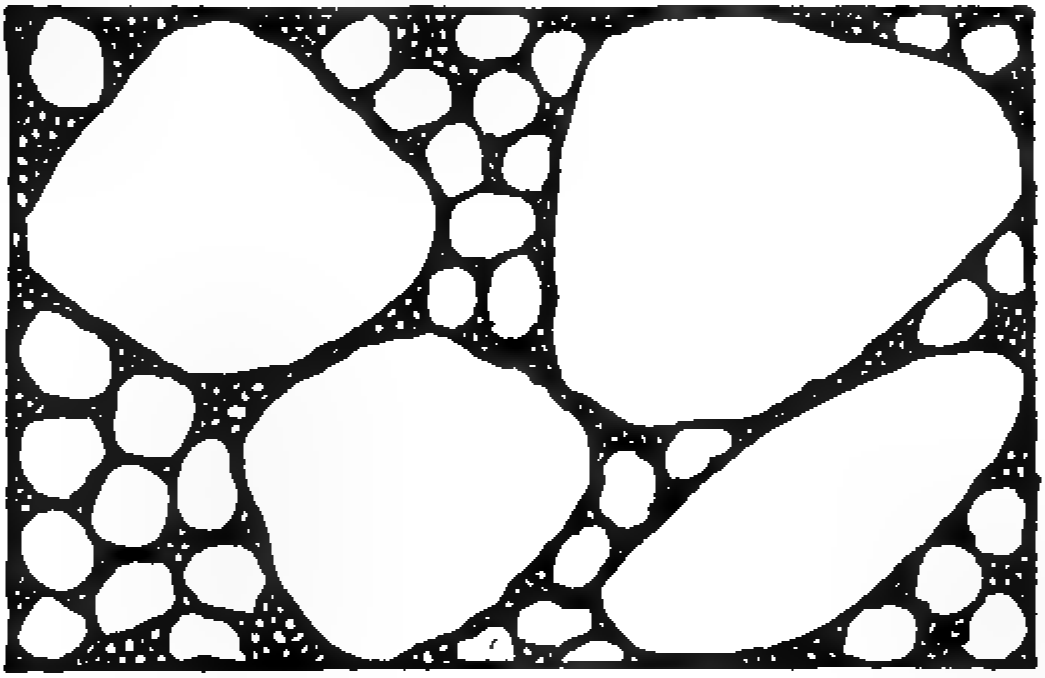

FIG. 60.—Large Stones with Voids filled with small Stones and Sand. (*See p.* 172.)

law relates directly to the difference between mortar and concrete. The substitution of stones for small masses of sand reduces the voids and consequently the quantity of cement required. Extending the principle to the fixing of proportions of sand and stone, it is evident that for maximum economy and equal strength there should be used the largest possible quantity of stone in proportion to the sand, the strength of concrete being often actually increased simply by substituting more stone for a portion of the sand. In the following table this is illustrated by tests selected from Mr. Fuller's 6-inch beam experiments, which are given in full on page 258.

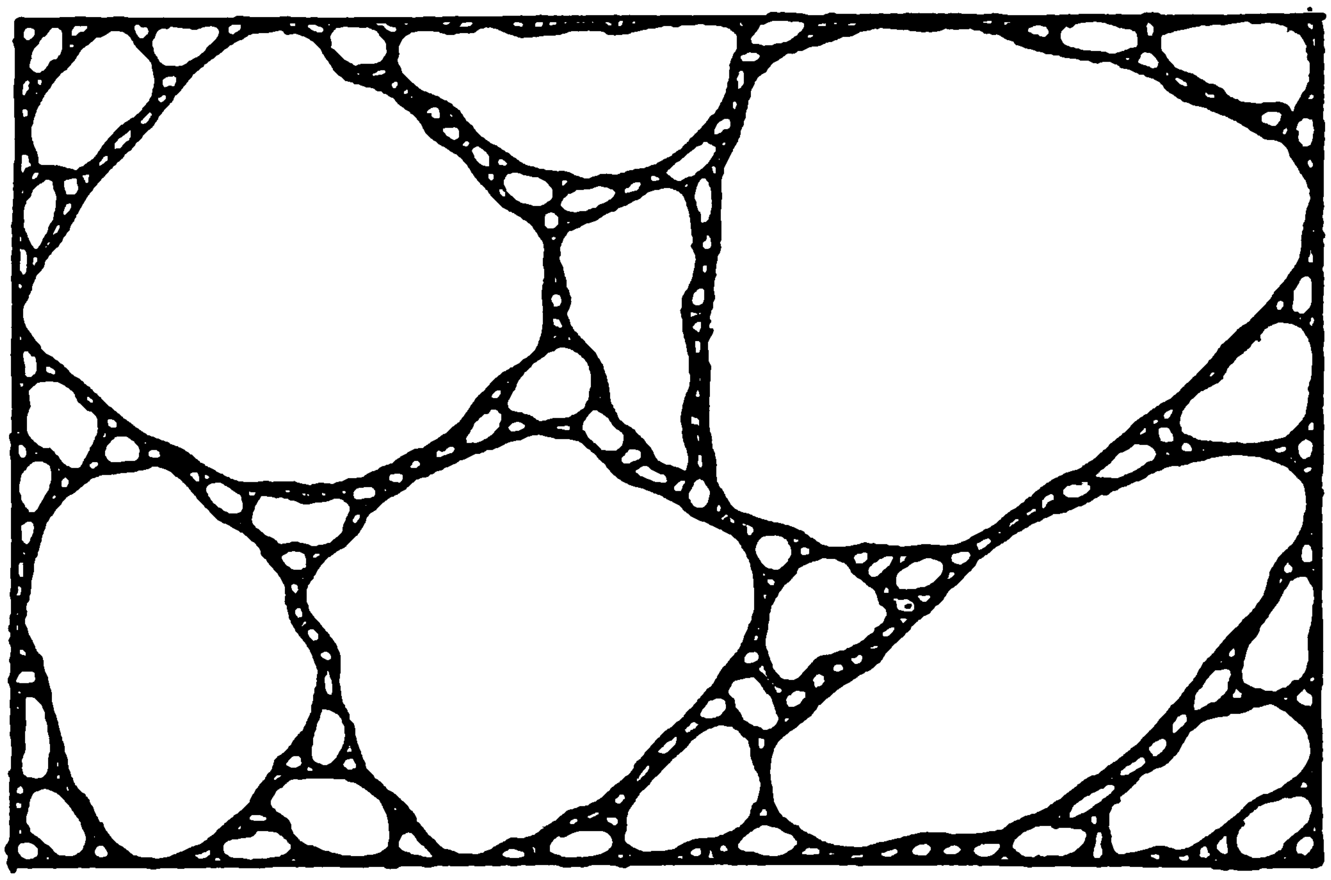

FIG. 61.—Large Stones, with Voids filled with medium sized Stones surrounded by smaller Stones and Sand so as to give Graded Mixture. (*See p.* 172.)

Relation of Strength of Concrete to Relative Proportions of Sand and Stone. (*See p.* 173.)

Proportions by weight of cement to total aggregate.	Proportions by weight of cement to sand and broken stone.	Modulus of Rupture lb. per sq. in.
1:6	1:1:5	504
1:6	1:2:4	439
1:6	1:3:3	355
1:6	1:4:2	210
1:6	1:6:0	93

The total amount of aggregate in each case is the same, namely, one part cement to 6 parts sand and stone, but the strength varies with the relative proportions of each, from 93 lb. to 504 lb.

(5) The discussion of Fuller's experiments on the relation of the best

practical mixture of sizes to a parabolic curve is given in Chapter XI, page 195.

Effect of Shape of Grain. (6) The fact that round grains, such as gravel, contain fewer voids than material with angular grains, such as broken stone, even if the particles in both are the same size, is proved from experiments in America and France. Mr. Allen Hazen states* that round grained water-worn sands have from 2% to 5% less voids than corresponding sharp grains of sand. Mr. Feret† also has studied the effect of the shape of the grain upon the density of sand, using in each case an artificial mixture of three sizes, with the following results:

Effect of Character of Sand Grains upon the Volume of the Sand. (See p. 174.)
BY R. FERET.

Nature of Sand	Shape of Grains	Actual solid volume per liter of sand Not shaken, liter	 Shaken to refusal, liter
Quartzite crushed in jaw crusher	Laminated	0.525	0.654
Crushed shells	Flat	0.557	0.682
Ground quartzite	Angular	0.579	0.726
Natural granitic sand	Rounded	0.651	0.744

The voids in each case are the complements of the figures given.

The conclusion to be drawn is that the real volume increases (and therefore the voids decrease) as the sand approaches the round form.

When experimenting upon gravels and broken stone Mr. Feret‡ separated each into three sizes which he called respectively:

G (coarse) passing holes of 6 cm. (2.36 in.) diameter and retained by holes of 4 cm. (1.57 in.) diameter;
M (medium) passing holes of 4 cm. (1.57 in.) diameter and retained by holes of 2 cm. (0.79 in.) diameter;
F (fine) passing holes of 2 cm. (0.79 in.) diameter and retained by holes of 1 cm. (0.39 in.) diameter.

Each size of broken stone loosely measured gave about 52% voids, and each size of gravel about 40% voids. The voids in the broken stone were reduced to 47%, the lowest result obtainable, by mixing G and F in about

*Twenty-fourth Annual Report, Massachusetts State Board of Health, 1892.

†Annales des Ponts et Chaussées, 1892, II, p. 32.

‡Annales des Ponts et Chaussées, 1892, II, p. 153.

equal parts with no M, and in the gravel to 34% with about 3½ parts of G to one part of F. These figures are of course directly applicable only

FIG. 62.—Standard Ottawa Sand, dry.* No. 20 to No. 30 Sieves. (*See p.* 175.)

FIG. 63.—Standard Ottawa Sand with 6% moisture.* No. 20 to No. 30 Sieves. (*See p.* 175.)

FIG. 64.—Natural Bank Sand.* No. 20 to No. 30 Sieves. (*See p.* 175)

FIG. 65.—Crushed Quartz.* No. 20 to No. 30 Sieves. (*See p.* 175.)

to the special materials which he studied, and do not apply to gravel or stone containing sand or dust.

Photographs of Sand. Photographs of three types of sand are shown in Figs. 62 to 65. Figures 62 and 63 are photographs of the Ottawa,

*Each sand has passed a No. 20 and been retained on a No. 30 sieve. Magnified 10½ diameters.

Illinois, bank sand screened to the size selected for the standard sand by the Committee of the American Society of Civil Engineers. They illustrate the effect of moisture upon the arrangement of the sand grains, which is more fully described below. Fig. 64 is an ordinary bank sand from Eastern Massachusetts which has passed through and been retained by the same screens as the Ottawa sand. Fig. 65 is a sample of crushed quartz sand, formerly the standard in the United States. The sands are all reduced by the same number of diameters. The Ottawa sand, Figs. 62 and 63, is apparently of finer grain than either the bank sand or the crushed quartz, but close inspection will show that its grains, very uniform in size, are of about the same diameter as the smallest grains in the other sands. In other words, all the grains correspond very closely to a No. 30 sieve, the lot of sand from which it was screened containing no larger particles.

Effect of Moisture on Sand and Screenings. (7) Moist sand occupies more space and weighs less per cubic foot than dry sand. This is directly contrary to what one would naturally suppose. Indeed, it is almost incredible that the addition of water can reduce the weight of any material. The statement is readily proved, however, by shoveling a small quantity of natural sand as it comes from the bank with, say, 3% or 4% of moisture into a measure and drying it. The sand will settle, leaving the surface much below the level of the top of the measure. The explanation of this apparent anomaly lies in the fact that a film of water coats each particle of sand and separates it by surface tension from the grains surrounding it. This is illustrated in Figs. 62 and 63, page 175, the grains of the moist sand being separated from each other by the film of water. Fine sand, having a larger number of grains, and consequently more surface area, is more increased in bulk by the addition of water than coarse sand. The volume of coarse broken stone and gravel is but slightly, if at all, changed by moisture, while small broken stone composed largely of particles of less than ½-inch diameter is affected like sand.

If a small quantity of water is poured into a vessel containing dry sand, the bulk is not increased because of the inertia of the particles, but if the sand after moistening is dumped out and then turned back into the vessel with a shovel or trowel, its bulk will be increased. On the same principle, a sand bank does not swell in bulk during a shower, but the effect of the moisture is shown in the excavated material as soon as it is loosened with the shovel, and therefore its loose measurement for concrete or mortar is effected.

The diagram in Fig. 66, plotted by Mr. Fuller* from experiments upon a single sample of natural sand mixed by weight with varying percentages of water, illustrates the effects of moisture upon the actual percentages of voids in sands loose and tamped. The volumes produced by varying degrees of compacting are located between the two curves. It is noticeable that both the loose and tamped sand increase in volume with the addition of water and reach a maximum with about 6% of water, then decrease, and finally, when saturated, return to slightly less than their original dry bulk. The same sand, it is seen, may contain from 27% to 44% of absolute voids, according to the percentage of water and the degree of compacting.

FIG. 66.—Percentage of Absolute Voids in a Natural Bank Sand containing Varying Percentages of Moisture. (*See p.* 177.)

The percentage of water by weight which will give the greatest bulk,—corresponding, of course, to the largest percentage of absolute voids,—varies with different sands from 5% to 8%.

The actual variation on different days in the percentage of moisture in a natural bank sand was found by the authors, in a series of experiments, to range from 1½% to 5¼% of the total weight, or from 2½% to 7¼% of the bulk of the moist sand. The sand, screened from a gravel bank in Eastern Massachusetts, ranged in coarseness from very fine to that which would pass a ⅛-inch mesh screen. The moist sample was taken from the pile the day after a shower, and weighed 84½ lb. per cubic foot, while the dryer sample, taken after a period of dry weather, weighed 107 lb. per cubic foot.

A sample of very fine sand which had been standing in a pile through the same shower contained 9½% of moisture by weight, corresponding to 13% by volume. Ordinary gravel, on the other hand, from which the sand had been screened, was found after a heavy rain to contain only 1.8% of moisture by weight, this being apparently the maximum quantity which it would hold.

**Engineering News*, July 31, 1902, p. 81.

The maker of concrete is especially interested in the influence of moisture upon the bulk of sand and upon its voids (1) because of its effect upon the actual measurement of sand used in construction work, and (2) because of its effect upon his experimental determinations of proportions.

Rather incomplete experiments of the authors tend to show that the actual effect of moisture upon the volume of sand used in concrete and mortar may often be less than would naturally be inferred from the various experiments cited, and depends largely upon the processes of handling the sand. For example, fairly dry sand (3% moisture) shoveled by laborers from the pile into the regular sand-measuring box weighed 454 lb., while after a rain, the sand (with 5% moisture) shoveled from the pile into the same box weighed 464 lb., that is, the moist sand was slightly heavier than the dry. Further handling reversed these relations, for on weighing these two sands in a half cubic foot measure, the moist sand, as we should expect, was lighter than the dry.

The explanation of this apparent discrepancy is undoubtedly due to the fact that as the rain which affected the moisture occurred after the sand had been excavated and piled near the mixing platform, its bulk, as suggested on page 176, was not affected. The laborers handling the moist sand took large shovelfuls and the arrangement of the grains was not greatly disturbed. If the sand had been excavated after the rain, the handling with shovels and dumping from the cart probably would have rearranged the grains so that the moist sand would have weighed less than the dry in the large measure as well as in the small box.

Mr. Feret* calls attention to the fact that mortars of nominally the same proportions are richer in winter than in summer because of the greater amount of moisture in the sand, which, by increasing its bulk, reduces the absolute volume of the grains in a unit of measure. On the other hand, mortars are leaner in dry than in damp weather because the sand has greater density when dry.

In the experimental study of sand for determining the proportions of cement to be used, the effect of moisture is exceedingly important. The voids in absolutely dry sand are certainly no criterion of its qualities for mortar, while a moist sand will give entirely different results on different days. The best that can be done, if the study can be pursued no further than void determination, is to select conditions as near as possible to the average, and after determining the voids, considered as air alone and also as space occupied by the air and moisture, to use the results as a basis for judgment, bearing in mind that the volume of paste made from 100 lb.

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*Annales des Ponts et Chaussées, 1892, II, p. 26.

of neat Portland cement, while varying largely with different brands, averages about 0.86 cubic feet, and that the volume of the additional water required for the sand (see pages 146 and 221) actually occupies space in the resulting mortar.

The most important conclusion to be drawn from the extreme variation in the same sand under different conditions is the impossibility of attaining results by the usual void experiments upon sand alone, which will be of accurate value in the consideration of mortar and concrete, and the practical necessity of employing methods such as are described by the authors in Chapter IX, page 138, or by Mr. Fuller in Chapter XI.

In the preceding paragraphs we have referred chiefly to the variation in the condition of the same sand. The importance of studying mortars rather than the sand alone is still further emphasized by the varying effect of moisture upon sands of different sizes. This is brought out very clearly in Mr. Feret's paper.[*] In studying the normal consistency of mortars he finds that not only every cement but also every sand has a definite percentage of water necessary to bring it to what may be called normal consistency. This he illustrates in the triangle shown in Fig. 67 (constructed as described on page 143), giving the "proportions of water (by weight) required for ground quartz sands of all granulometric composition." It is evident from the diagram that coarse sands,[†] G, require 3% by weight of water, medium sands, M, 9%, and fine sands, F, 23%, while mixtures of the three sizes require intermediate percentages.

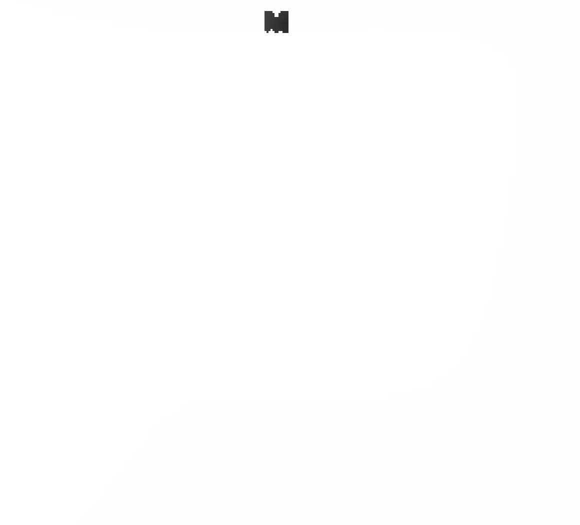

FIG. 67. — Percentages of Water Required to Gage Ground Quartz Sand of all Granulometric Compositions. (*See p.* 179.)

Compacting of Broken Stone and Gravel. Since concrete is usually compacted by ramming or lubrication of semi-liquid mortar, the density or the percentage of voids in compacted material is an important function. The statement has been made frequently that the aggregate compacts more when rammed in concrete than when rammed dry or merely moistened with water, because the mortar acts as a lubricant. Experiments by the authors indicate that broken stone under the same ram-

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[*] Annales des Ponts et Chaussées, 1892, II.

[†] The sizes of screens defining coarse, medium, and fine sands are given on page 142.

ming will compress on the average 1% more when it is moistened than when dry, and that an amount of mortar sufficient to lubricate without filling the voids produces no further reduction in volume. For example, a volume of broken stone mixed with 20% of mortar and rammed in 6-inch layers produced a volume exactly equal to that of the rammed broken stone which had been merely moistened.

Further experiments, partially outlined in the table on page 171, upon gravel and also upon varying sizes and mixtures of trap rock from two quarries, the one producing a soft and the other an exceedingly hard stone, lead to the conclusion that with stones of the same general structure, the percentage of reduction in volume by similar ramming in 6-inch layers is quite uniform, irrespective of the actual sizes of the particles, their relative sizes, the percentage of voids, and, within certain limits, the degree of hardness. On the other hand, the method of ramming the same stone will very largely affect the amount of compacting. Broken stone of the nature of trap, whether hard or soft, was found to compact when spread in 6-inch layers about 14% either under light ramming or shaking the measure, and about 21% under heavy ramming. In actual concrete work this large reduction of volume is of course seldom reached, because imperfect mixing and the necessary coating of the particles require a larger percentage of mortar than will just fill the voids of the rammed stone, and the bulk of concrete is usually greater than that of the original stone.

Screened gravel spread in 6-inch layers and unconfined, compacted about 12% under either light or heavy ramming.

These percentages of compacting are based upon the loose measurement of the material as thrown by a laborer into a barrel or box measure. Rehandling a material like broken stone as it comes from the crusher tends to mix particles of unequal size and therefore to compact it very slightly. In one case a screened stone fresh from the crusher compacted 1% when rehandled once, and an additional 1% when rehandled the second time.

It is interesting to note that the method of shoveling broken stone into a measure has but slight effect upon its shrinkage; for example, a lot of stone thrown with force into an inclined barrel occupied a space scarcely appreciably less than when very carefully and lightly placed. On the other hand, dropping from a considerable height does affect the volume, for Mr. Desmond Fitzgerald* states that broken stone dropped 12 feet into a car shrank to a volume 7% less than when it was measured in a box.

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*Transactions American Society of Civil Engineers, Vol. XXXI, p. 303.

Sand, unlike stone, is largely affected by the manner of shoveling and the size of the receptacle.

Compacting of Sand. The degree of compacting of sand is largely dependent upon the percentage of moisture which it contains. The dry sand shown in diagram in Fig. 66, page 177, when thoroughly tamped compacted from 34% to 27% voids or 9.6% in volume,* the sand with 6% moisture from 44% to 31% voids or 18.8% in volume, and the saturated sand from 33% to 26½% voids or 8.8% in volume.

Attention is called by Mr. Feret to the fact that the measurement of the weight of a given sand depends not only upon the quantity of moisture in it, but also upon the depth of the box which is used for the measure, the quantity of sand introduced at a time, — that is, the size of a shovelful, — the height from which it falls, the amount of shaking, if any, given to the box during filling, the amount of compacting given to the mass when leveling it off, and the smoothness of the surface left. As an illustration of the difference due to the method of placing in the measure, the authors found that a certain coarse sand shoveled into a pail about as a laborer would fill a measure weighed 88.9 lb. per cubic foot, while the same sand carefully poured into the pail weighed 83.3 lb. per cubic foot.

DEFINING COARSENESS OF SAND BY ITS UNIFORMITY COEFFICIENT

The size of a sand is best indicated by what is termed its uniformity coefficient. This gives an idea of the actual variation in the size of the particles, and thus affords a means for comparing sands in different localities. A sand which is termed *coarse* in one section of the country is often considered *fine* in another.

To find the uniformity coefficient of a sand, screen it into at least five sizes, determine the percentage by weight of each size, and plot the mechanical analysis curve as described on page 190, and illustrated in Fig. 72, page 194. Then divide the diameter of the particles represented by the point at which the curve of the sand crosses the 60% horizontal line by the diameter of the particles where the curve crosses the 10% line. The quotient is the uniformity coefficient.

As an illustration of the value of the uniformity coefficient (u. c.) for different sands, reference may be made to the three mechanical analysis curves in Fig. 72, page 194. The curve of the coarse sand crosses the

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*Ratio of compacting $= \frac{0.34 - 0.27}{1.00 - 0.27} = 0.096$

horizontal 60% line at the ordinate corresponding to a diameter of 0.117 inch, and the 10% horizontal line at ordinate 0.023 inch. Its uniformity coefficient and similarly the uniformity coefficients of the other sands are as follows:

			Uniformity Coefficient
Coarse sand	$\frac{0.117}{0.023}$	=	5.1
Medium sand	$\frac{0.038}{0.009}$	=	4.2
Fine sand	$\frac{0.018}{0.008}$	=	2.2

In general, it may be said that a sand with a uniformity coefficient above 4.5 is a good coarse sand for concrete work, and in comparing different natural sands the one having the highest uniformity coefficient may be considered the best.

As in ordinary bank sands the size of the particles at the 10% line (which is termed the effective size,* e. s.) does not greatly vary, the diameter at the 60% line alone is a very good indication of the coarseness of the sand. A knowledge of the effective size and the uniformity coefficient of any sand enables one accustomed to mechanical analysis diagrams to form a picture of its character.

Mr. Allen Hazen,† who first used these terms in the examination of filter sand, states with reference to the percentage of voids or "open space" in compacted sand corresponding to different coefficients:

> A rough estimate of the open space can be made from the uniformity coefficient. Sharp-grained materials having uniformity coefficients below 2 have nearly 45 per cent. open space as ordinarily packed; and sands having coefficients below 3, as they occur in the banks or artificially settled in water, will usually have 40 per cent. open space. With more mixed materials the closeness of packing increases, until, with a uniformity coefficient of 6 to 8, only 30 per cent. open space is obtained, and with extremely high coefficients almost no open space is left.

For loose sand at least 10 should be added to these percentage values.

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* The effective size itself is of considerable value for comparison of sand for filters, but not for concrete.

† Twenty-fourth Annual Report of State Board of Health of Massachusetts for 1892.

CHAPTER XI

PROPORTIONING CONCRETE

BY WILLIAM B. FULLER.*

IMPORTANCE OF PROPER PROPORTIONING

Upon large or important structures it pays from an economic standpoint to make very thorough studies of the materials of the aggregates and their relative proportions. This fact has been seriously overlooked in the past, and thousands of dollars have sometimes been wasted on single jobs by neglecting laboratory studies or by errors in theory. Since cement is always the most expensive ingredient, the reduction of its quantity, which may very frequently be made by adjusting the proportions of the aggregate so as to use less cement and yet produce a concrete with the same density, strength and impermeability, is of the utmost importance.

As an example of such saving, the ordinary mixture for water-tight concrete is about 1: 2½: 4½, which requires 1.37 barrels of cement per cubic yard of concrete. By carefully grading the materials by methods of mechanical analysis the writer has obtained water-tight work with a mixture of about 1: 3: 7, thus using only 1.01 barrels of cement per cubic yard of concrete. This saving of 0.36 barrels is equivalent, with Portland cement at $1.60 per barrel, to $0.58 per cubic yard of concrete. The added cost of labor for proportioning and mixing the concrete because of the use of five grades of aggregate instead of two was about $0.15 per cubic yard, thus effecting a net saving of $0.43 per cubic yard. On a piece of work involving, say, 20 000 cubic yards of concrete such a saving would amount to $8600.00, an amount well worth considerable study and effort on the part of those in responsible charge.

The most practical method yet discovered by the writer for accurately determining the proportions of each material is by mechanical analysis of the aggregates described on page 187. Volumetric synthesis or proportioning by trial mixtures (p. 209) is another method which is sometimes useful.

Proportioning by the determination of voids, discussed on page 210, and the arbitrary selection of proportions such as 1: 2: 4, 1: 2½: 5, 1: 3: 6, 1: 4: 8, according to the nature of the work, with no reference whatever

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*The authors are indebted to Mr. Fuller for the material for this chapter.

to the character of the materials, as discussed more fully on page 212, are the methods which have been most commonly employed.

INACCURACY OF PRESENT METHODS OF PROPORTIONING

The practice of determining the proportions by finding the quantity of water which may be poured into the voids of a unit volume of stone and selecting a volume of sand equal to the volume of the quantity of water is one not to be recommended.

The chief inaccuracy of this method of basing the proportions of the finer materials of a concrete mixture upon the water contents of the voids in the larger is due to the difference in compactness of the materials under varied methods of handling, and to the fact that the actual volume of voids in a coarse material may not and usually does not correspond to the quantity of sand required to fill the voids, and that therefore the common method of proportioning by basing the volume of sand or of mortar upon the volume of water which can be poured into the broken stone leads to false conclusions. The reasons for this inaccuracy are chiefly because the grains of sand thrust apart the particles of stone, and because with most aggregates a portion of the particles of sand or fine screenings are too coarse to enter the voids of the coarsest material.

Even in a mass of stones of uniform size many of the separate voids are much smaller than the particles. If we have, then, a mass of gravel ranging from fine to coarse or a mass of crusher-run broken stone, even with the finest sand or the dust screened out of them, the individual voids are many of them so small that a large number of the particles of natural bank sand will not fit into them, but will get between the stones and increase the bulk of the mass. On account of this increase in bulk, even with thorough mixing more sand is required than the actual volume of the voids in the coarse material. The separation of the particles of stone by the sand is illustrated in the mixture shown in Fig. 2, page 15.

To illustrate this important principle, an extreme example may be cited. Suppose that we have a mixture in equal parts of 1-inch stone and ⅞-inch stone. By the usual method of reasoning employed in proportioning concrete, if the 1-inch stone has 50% voids, we should require a volume of ⅞-inch, equal to 50% of the volume of the 1-inch stone, in order to fill the voids in the latter. The absurdity of this is apparent, because the two stones are so near a size that the smaller cannot fit into the voids of the latter, and the bulk of the mixture is inappreciably less than the sum of the separate volumes, that is, the mixture still has nearly 50% voids. The principle is just as true, although the total

effect is less, if we consider it with reference to the finer particles of the gravel or the crusher-run broken stone and the sand or fine screenings which are to be introduced to fill the voids. The sizes of many of the particles of the latter are so nearly equal to the sizes of the smallest particles of the coarse material that they increase the total bulk instead of reducing the voids. They also get between the surfaces of the stone particles and prevent the stones touching each other.

We might conclude from the above that the best concrete can be made with a coarse stone of uniform size and a sand whose particles are all small enough to fit into its voids; in fact, this is the conclusion reached by the advocates of broken stone of uniform size in preference to crusher-run stone. Both theory and experiments, however, prove that the deduction is incorrect, because the smallest percentage of voids occurs when the mixture is graded.*

The point, however, which is to be emphasized is the inaccuracy of determining the exact volume of sand or mortar by simply measuring the water contents of the voids in the coarse aggregate.

The selection of the proportion of cement by determination of the water contents of the voids in sand is even more inaccurate than the proportioning of sand to stone by void measurement. The varying effect of moisture on the sand so influences the volume of the voids that their determination is chiefly important as an aid to the judgment, and as a matter of fact, although in practice the quantity of cement is supposed to depend upon the volume of voids in the sand, it is customary to select a definite relation of cement to sand varying according to the character of the construction from 1:2 to 1:4, recognizing, however, that fine sand — and fine sands in an ordinary state of moisture will almost always have the distinguishing characteristic of a lighter weight per cubic foot than coarse sands and a consequently larger percentage of voids — requires more cement for equivalent strength.

If the work is too small to warrant a thorough study of the materials by mechanical analysis or volumetric synthesis, or some other scientific method, it is evident from the above discussion that it is nearly as accurate to determine the proportions by arbitrary selection (see p. 212) as by a study of voids.

PRINCIPLES OF PROPER PROPORTIONING

The principles underlying the correct proportions of the materials of concrete are the same as those for mortar, namely, that the mass when

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*See p. 171 and Figs. 59, 60, and 61, pp. 172 and 173.

compacted shall have the greatest possible density. In order, therefore, to obtain a knowledge of correct proportioning it will be best to first study the general conditions which are known to affect density.

Perfect spheres of equal size piled in the most compact manner theoretically possible leave but 26% voids. If the spaces between such a pile of equal-sized perfect spheres were filled with other perfect spheres of diameter just sufficient to touch the larger spheres, it would take spheres having relative diameters of 0.414 and 0.222 of the larger spheres, and the voids in the total included mass would be reduced to 20%. Using in this same manner smaller and smaller perfect spheres, it is conceivable that the voids could be reduced to so low a per cent of the total mass and to a size so small as to be only in a capillary form, and thus prevent the passage of water. This is assuming that every particle is placed exactly in its assigned place, but it is inconceivable that such an arrangement should take place under practical conditions, and in fact numerous trials by the writer with large masses of equal-sized marbles have demonstrated that they cannot be poured or tamped into a vessel so as to give less than 44% voids.

If equal quantities of spheres of, say, three sizes are mixed together, the per cent of voids in the total mass immediately increases, becoming about 65%, due probably to the smallest spheres getting between and forcing apart the largest. If, however, the containing vessel is continually shaken and the spheres stirred around, the smallest spheres will gradually all gravitate to the bottom and the largest to the top and the amount of voids in the total mass will again approach 44%. If a large number of different sized spheres are used, employing an increasingly large number of the smaller sizes so that each larger size may be said to be wholly surrounded by the next smaller size, the voids remain the same, no matter what the shaking, and will be considerably less than 44%.

With ordinary stones and sands the same law holds as with perfect spheres except that they do not compact as closely, and the percentage of voids under comparable conditions is larger, varying with the degree of roughness and other features of the stones and sands used for the experiments. The addition of water to stones and sands does not affect their compactness if enough water is added to flush to the surface. Where less water than this is added, air is entangled by the particles of fine sands, and the compactness of the mass is decreased.

When dry cement is added to a dry aggregate of stone and sand it acts in the same manner as fine sand, and for obtaining the greatest density with dry cement the cement must replace an equivalent amount of fine sand.

The theory of a concrete mixture is well stated by Mr. Feret* as follows:

> The problem of making the best concrete is thus reduced to the selection of a mixture of materials whose granulometric composition† corresponds to the maximum of density, since when this composition is known, absolute volumes of cement may be substituted for equal absolute volumes of fine sand and vice versa, so as to vary the strength as desired while the density remains the same.

In other words, having mixed dry, inert materials in proportions necessary for greatest density, a portion of the grains of the very finest aggregate (that is, the finest particles of sand or dust) may be replaced by a corresponding quantity of cement to the extent required for the desired strength. This is not strictly true for concrete mixtures, because, when water is added to dry cement, the cement particles are separated from each other by the surface tension of the film of water, and it is no longer possible to obtain as dense a mixture as will be given by the dry mixture.

The density of concrete therefore has been found to depend upon the varying degree of roughness of the stone and sand, the relative sizes of the diameters of the stone, sand and cement, and the amount of water used.

The fineness of the cement particles and the amount of water to be used are determined by questions discussed elsewhere, and we have to deal here only with the proportioning of the sand and stone.

MECHANICAL ANALYSIS

Mechanical analysis consists in separating the particles or grains of a sample of any material, — such as broken stone, gravel, sand or cement, — into the various sizes of which it is composed, so that the material may be represented by a curve (see Fig. 70, p. 192) each of whose ordinates is the percentage of the weight of the total sample which passes a sieve having holes of a diameter represented by the distance of this ordinate from the origin in the diagram.

The objects of mechanical analysis curves as applied to concrete aggregates are (1) to show graphically the sizes and relative sizes of the particles; (2) to indicate what sized particles are needed to make the aggregate more nearly perfect and so enable the engineer to improve it by the addition or substitution of another material; and (3) to afford means for determining best proportions of different aggregates.

To determine the relative sizes of the particles or grains of which a given

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*Chimie Appliquée, 1897, p. 523.

†Proportioning of sizes.

sample of stone or sand is composed, the different sizes are separated from each other by screening the material through successive sieves of increasing fineness. After sieving, the residue on each sieve is carefully weighed, and beginning with that which has passed the finest sieve, the weights are successively added, so that each sum will represent the total weight of the particles which have passed through a certain sieve. The sums thus obtained are expressed as percentages of the total weight of the sample and plotted upon a diagram with diameters of the particles as abscissas and percentages as ordinates.

The method of plotting and the uses of the curves thus obtained are more fully described in the pages which follow.

Sieves and Other Apparatus. Fig. 68 illustrates a convenient outfit for such a mechanical analysis as above described, consisting of a set of sieves, an apparatus for shaking the sieves, and scales for weighing. A standard size of sieve is 8 inches in diameter and 2½ inches high. Sieves with openings exceeding 0.10 inches are preferably made of spun hard brass with circular openings drilled to the exact dimensions required. Sieves with openings of 0.10 inch and less are preferably of woven brass wire set into a hard brass frame. Woven brass sieves are made for many purposes, and are sold by numbers which approximately coincide with the number of meshes to the linear inch. As the actual diameter of the hole varies with the gage of wire used by different manufacturers, every set of sieves must be separately calibrated.

An approximate idea of the diameters of holes which may be expected in commercial sizes of sieves is presented in the following table, which is sufficiently exact to serve as a guide to the purchase of the sieves:

Commercial No. of sieve.	Diameter of hole in inches.	Commercial No. of sieve	Diameter of hole in inches.
10	0 073	60	0.009
15	0.047	74	0.0078
16	0.042	100	0.0045
18	0.037	140	0.003625
20	0.034	150	0.00325
30	0.022	170	0.0031
35	0.017	180	0.00306
40	0.015	190	0.0028
50	0.011	200	0.00275

For separating particles smaller than those passing through a No. 200 sieve, recourse must be had to processes of elutrition which have been developed to great precision by soil analysis chemists.*

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*See page 85.

FIG. 68. — Mechanical Analysis Sieves and Shaker. (*See p.* 188)

In selecting the right series of sieves to purchase, first decide on the limiting diameters, say, from 3.00 inches to No. 200 = 0.00275 inches. Then decide on the total number of sieves, say, twenty. Look up the logarithm of 3.00 and of 0.00275 and by proportion find eighteen other logarithms between these having equal differences between each. Look for the number corresponding and take the nearest commercial sieve giving this diameter. The diameters of holes exceeding 0.10 inch can be made as required. A convenient set of twenty sieves, — ten for stone, which give the diameter of the holes in inches, and ten for sand, giving the commercial number (see p. 188), — is as follows:

Stone Sieves inches.	Sand Sieves Commercial No.
3.00	10
2.25	15
1.50	20
1.00	30
0.67	40
0.45	60
0.30	74
0.20	100
0.15	150
0.10	200

After the sieves are obtained it is necessary that they should be very carefully calibrated to ascertain the average diameter of the mesh. This should be done by averaging the diameters of the openings measured in two positions at right angles to each other, as the meshes of commercial sieving are not exactly square. Sieves having meshes exceeding 0.10 inch are most conveniently calibrated by ordinary outside calipers; those having meshes of less diameter, by a micrometer microscope.

When many analyses are to be made, it is convenient to have a printed cross section form, such as is shown in Fig. 69, p. 191, with appropriate spaces for filling in the number of the analysis, description of the material, location of the work, and other facts relating to the material.

Plotting Analysis Curves. For those who are unfamiliar with mechanical analysis a detailed explanation of the method of locating the curve is here given. The method can best be understood by referring to the diagrams of typical materials which are also of practical interest as illustrating the curves which may be expected in special cases.

Fig. 70, p. 192, represents a typical mechanical analysis of crusher-run micaceous quartz stone which has been run through a ¼-inch revolving screen so as to separate particles finer than ¼ inch, that is the dust, for use with sand.

For a sample of stone, which may be taken by the method of quartering

Fig. 69. — Blank Form for Mechanical Analysis Diagram. (*See p.* 190.)

described on page 280, 1 000 grams is a convenient quantity for 8-inch diameter sieves 2½ inches in depth, and also permits of easy reduction from weights to percentages. To obtain the analysis shown in Fig. 70, the sample of stone is placed in the upper (coarsest) sieve of the nest of stone sieves given on page 190, and after 1 000 shakes the nest is taken apart, and the quantity caught on each sieve is weighed. The results obtained in the particular case under consideration are illustrated in the following table, which shows the method of finding the percentages:

FIG 70. — Typical Mechanical Analysis of Crusher-Run Micaceous Quartz Stone. (*See p.* 192.)

Results of Screening Samples of Stone of Fig. 70.

Size sieve inches.	Retained in each sieve* grams.	Amount finer than each sieve grams.	Percentage finer than each sieve %
0.10	8	0	
0.15	11	8	1
0.20	8	19	2
0.30	72	27	3
0.45	123	99	10
0.67	235	222	22
1.00	344	457	46
1.50	199	801	80
Total,	1000		

*In practise this column is not required, the weights in the next column being obtained directly by placing each successive residue on the scale pan with that already weighed.

The various percentages are plotted on the diagram and the curve drawn through the points. The vertical distance from the bottom of the diagram

to the curve, that is, the ordinate at any point, represents the percentage of the material which passed through a single sieve having holes of the diameter represented by this particular ordinate. Since the percentage of material passing any sieve is always the complement of the percentage of grains coarser than that sieve, the vertical distances from the top of the diagram down to the curve represents the percentages which would be retained upon each sieve if employed alone. For example, taking 1.25, 62%, the distance from the bottom of the diagram, represents the percentage of material finer than 1¼ inch diameter, and 38%, the distance down from the top of diagram, represents the percentage coarser than 1¼ inch.

Fig. 71 represents a typical analysis of crushed trap rock which has been

FIG. 71.—Typical Mechanical Analysis of Crushed Trap Rock Separated into Three Sizes by Revolving Screens having 3, 1½, ¾ and ½ inch perforations. (*See p.* 193.)

separated into stone of three sizes and dust, by a revolving screen 2 feet 6 inches in diameter and 12 feet long set on a slope of 1 foot 9 inches. This was made up of four sections having respectively 3, 1½, ¾ and ½ inch perforations. The curves not only show the sizes of trap rock which ordinarily pass through crusher screens of given diameter of hole, but also illustrate how inefficient the screening process may be. For example, if the sizes of the particles had corresponded exactly to the diameters of the holes and the screening had been more perfectly done, we should have had curves whose general direction and location is shown by the dotted lines No. 2_1, No. 3_1, and No. 4_1, that is, for example, No. 3, since it represents stone which passes a 1½ inch screen and which is retained on a ¾ inch screen, should occupy a position between the ordinates representing 1.50 and

0.75 diameters. If the stone had rumbled longer in the screen because of flatter slope or screen sections of greater length, the curves would have approached more nearly to these dotted lines.

Typical curves of a fine, a medium well graded, and a coarse sand are shown in Fig. 72. For convenience in plotting, the horizontal scale is ten times greater than that of Figs. 70 and 71, the diagram showing diameters ranging from 0 to 0.200 inches diameter. The "granulometric composition" of these sands may be determined if desired by reference to page 149.

FIG. 72.—Typical Mechanical Analyses of Fine, Medium, Well Graded and Coarse Sands. (*See p.* 194.)

The mechanical analysis of crusher dust is apt to vary between the curves of fine sand and medium sand which are shown in Fig. 72.

PROPORTIONING THE AGGREGATE BY MECHANICAL ANALYSIS*

In the year 1901 the writer, through the permission and assistance of Mr. E. LeB. Gardiner, Vice-President, and Mr. J. Waldo Smith, Chief

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*Mr. Fuller's method of proportioning the materials so that their mixture will form a curve approaching a parabola appears, on its face, to conflict with Mr. Feret's conclusion (see p. 147) that the best mixture of sand and cement for mortar is made up of coarse and fine grains only with no intermediate grains. For sand mortars, Mr. Feret's methods are undoubtedly more exact than Mr. Fuller's, but for a concrete mixture the conditions are different, and as we have stated on page 172, more than two sizes of materials are theoretically necessary for obtaining the densest mixture. In practise, too, all classes of materials are more or less varied, and experiments show that the particles will best fit into each other if the sizes are graded. The best proof of the practical efficiency of Mr. Fuller's method lies in the fact that he has employed it day after day for determining the proportions of the aggregate for concrete used in constructing thin, water-tight walls. The proportions used by him for such work are about 1: 3: 7, whereas for water-tight

Engineer, of the East Jersey Water Company, was enabled to make an extended series of experiments on the comparative strengths of different proportions of concrete aggregate. Many mixtures of different proportions were made up into beams, their curves of mechanical analyses drawn as explained above, and the strength of the beams determined by breaking tests.*

The experience which the writer has had and the various experiments which he has made indicate that concrete which works the smoothest in placing and gives the highest breaking strength for a given percentage of cement is made from an aggregate whose mechanical analysis, taken after mixing the sand and the stone, forms a curve approaching that of a parabola, with its beginning at zero coördinates (0) and passing through the intersection of the curve of the coarsest stone with the 100% line, that is, passing through the upper end of the coarsest stone curve.

This, therefore, furnishes a very convenient method for determining the best proportions of any given materials to use, and also a means of ascertaining by inspection which of two materials is better adapted for the work, and whether the sizes of stone should be changed in order to improve the mixture. Mechanical analysis curves of samples of each of the selected materials are plotted upon a diagram, and a parabola is drawn on the same diagram through zero coördinates (0) and the intersection of the 100% line of the diagram and the ordinate corresponding to the largest diameter of stone particles. By calculation or inspection the percentages of each of the materials are determined, which, mixed together, will give a resultant mechanical analysis curve most nearly coinciding with the parabola.

The method of determining the percentages of the different materials is extremely simple, and the proportions may be calculated with the assistance of a slide rule in a few minutes time. Since, however, the processes followed are somewhat unusual, it seems wise to present what may appear like a very elementary explanation, as the intermediate steps must be

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construction where the materials are not scientifically graded 1:2:4 mixtures are commonly used.

The method is exact and scientific and not "rule-of-thumb." The nature of the materials and their variation from hour to hour makes great refinement unnecessary, so that an accuracy of, say, 2% or 3% in the percentages are all that is necessary in practice.

The experiments upon which Mr. Fuller's theory and practice are based were made with mixtures in which the coarsest stone was about 3 inches in diameter. It has not yet been clearly demonstrated whether a curve of the exact form of the parabola applies to all classes of materials, but the general method of analyzing the materials and combining the curves is undoubtedly applicable whether the resultant curve is a parabola or a curve of different form, and if it should ultimately be found that different materials require curves of different shape, Mr. Fuller's general principles and methods still hold. — *The Authors.*

*The results of these tests are presented in the table on pages 258 and 259.

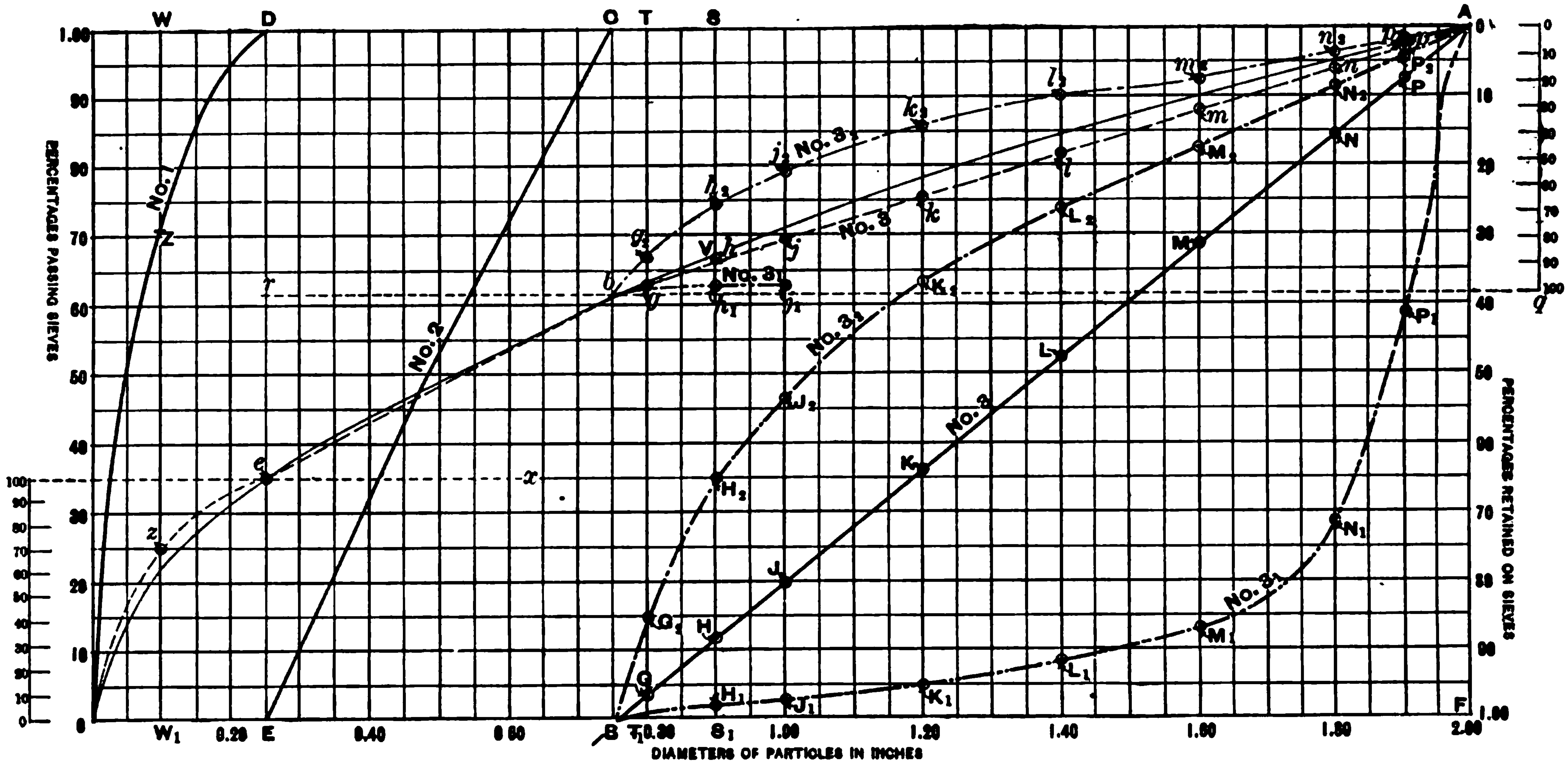

FIG. 73.—Diagram Illustrating Method of Combining Curves which do not Overlap. (*See pp.* 197 *to* 203.)

thoroughly understood although they need not be followed in actual practise.

Methods of Combining Curves to Form an Approximate Parabola. Before taking up practical examples given on pages 205 and 207 illustrating the method of determining proportions of the various ingredients, and of studying graphically the character of the curves, it will be necessary to explain in detail by simpler cases the principles governing such graphical combinations, so that the inexperienced computer may clearly see the meaning of the curves and combinations of curves by a glance at a diagram. In practice the reduction of the material curves is very simple and the percentages are obtained more by inspection than by mathematical calculation, although the method is mathematically correct.

Case I. Curves which meet, but do not overlap. In Fig. 73 are shown three curves, No. 1, No. 2, and No. 3, representing ideal grades of sand and stone, which may be combined in such proportions that the curve of the mixture will form a nearly perfect parabola. The problem requires the determination of the percentages of each of the three materials which when combined will form a mixture whose curve is nearly a parabola. In order to prove that the percentages found will produce the resultant curve, and also to illustrate the theory of the mixture, the resultant curve will be first plotted and described in a very elementary manner, and afterwards by the method of ratios which would be employed in practise.

Curve No. 3 represents a material all of whose particles will pass through a sieve having holes 2.00 inches diameter and all of whose particles will be retained on a sieve having holes 0.75 inches diameter. Stone represented by curve No. 2 lies between diameters 0.75 and 0.25 inches, while the material of curve No. 1 is all finer than 0.25 inches, that is, is all under ¼ inch. Curves No. 3_1 and No. 3_2 are referred to later.

The parabola *OebA* is first plotted.* We assume now that the best theoretical mix of materials lies in this parabolic curve. This is equivalent to saying that the best theoretical mixture of materials is such, that at any ordinate or vertical line cutting the parabola, the proportion or percentage

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*CONSTRUCTION OF THE PARABOLA.

D = largest diameter of stone
d = any given diameter
P = per cent. of mixture smaller than any given diameter

The equation of the parabola is

$$d = \frac{P^2 D}{10\,000}$$

The parabola can be constructed in any of the numerous ways given in text books, the writer finding it easiest to use a slide rule. Set D on the B scale of the rule opposite 100 on D scale, read any value of d on the B scale opposite any corresponding value of P on the D scale.

of the ordinate below the intersection represents the percentage by weight of the mixed materials which passes a sieve the diameter of whose openings corresponds to the given ordinate, and the percentage above the curve represents that percentage which is too large to pass through this sieve. The parabola shows, for example, that 87% of the mixture of materials should pass a 1.50-inch sieve, 71% should pass a 1-inch sieve, 49% a ½-inch sieve, and so on.

We may now take up the stone curves in succession to determine what percentage by weight of each should be used, so that when they are combined, the mixture will be as nearly as possible like that called for in the parabola.

The chief difficulty in the method of determining the percentages of each material lies in combining the individual curves so as to form a single curve which represents the mixture. This involves drawing on the same piece of paper two different lines, each of which exactly represents the composition of the same lot of stone, that is, the exact per cent. of each size of stone in the lot. For example, as is explained below, on Fig. 73, lines *BKA* and *bkA*, each accurately represents the percentage composition of the same batch of stone, namely, No. 3, and the full meaning and value of these diagrams cannot be understood until it is clear how the same values can be accurately represented on the same diagram by two such totally different curves.

In the first place it is seen that the ordinates, that is, the vertical lines in the diagram, are divided into 100 parts representing percentages. It is clear, therefore, as the divisions are relative, that the diagram would accomplish the same results and curves could be drawn accurately representing the percentages passed and retained by the different sieves, whether the distance from 0 to 100 on the ordinates were, say, three times as large as it is, or whether it were only ¼ or ⅛ of the present length. All that is needed is to divide these vertical lines, whether they are long or short, into 100 parts and let each division represent 1%.

Referring now to Fig. 73, the percentage composition of the No. 3 lot of stone is represented by line *BKA*. This lot of stone contains no stone smaller in diameter than 0.75 inches and none larger than 2.00 inches. Running vertically upward from *B* on the 0.75 inch line to *b* where it crosses the parabola, we see that the parabola from *b* to *A* also represents a lot of stone none of which is smaller than 0.75 inches and none larger than 2.00 inches, and we can look upon this lot of stone for the moment as entirely separated from the rest of the mixture which the whole parabola represents. If we wish to find the exact percentages of the various sizes

of stone which are in the portion or lot represented by the portion of the parabola from b to A, all that is necessary is to draw the horizontal line rq through the point b, then divide the vertical distance from A to rq into 100 parts, so as to obtain a new set of horizontal lines or abscissas representing percentages. Now if we start at the base line rq and follow up any one of the vertical lines or ordinates until it meets the parabola, and then follow horizontally to the right along the line which intersects the parabola at the same vertical line or ordinate point, the reading on the new smaller percentage scale will give us the per cent. of stone in the lot bA which is larger than the diameter represented by this ordinate, etc. For example, taking intersection of 1.00 ordinate with the parabola and running across we find that 75% of the lot is coarser than 1 inch diameter.

It is desirable to see how nearly the stone in lot No. 3 agrees with the theoretical lot of stone called for by section bA of the parabola. In practice, the comparison may be made most readily by ratios with the aid of the slide rule, as is described more fully below, but the reasoning will be more clearly understood if the plan described in the last paragraph is followed.

Taking first curve No. 3 we may redraw it on the same smaller scale as the portion of the parabola bA is drawn, that is, it may be constructed on rbq as a base line instead of on the zero coördinate BF. Since the vertical per cent. line between q and A has been divided into 100 parts, this section of the diagram may be used instead of the original per cent. divisions extending from A to F. A piece of paper the length of Aq may be divided into 100 parts and placed with its upper or 0 end in line with the upper line CA of the diagram. The vertical distance from the line CA to the various points G, H, J, K, etc., may be read by the eye and replotted, — with the assistance of the small scale, — as g, h, j, k, etc.

It is evident then that the broken line $bghjKA$ represents (referring to the small percentage scale Aq) lot No. 3 of stone as accurately as line $BGHJKA$ represents the same lot of stone referring to the larger percentage scale AF.

Stone curve No. 3, however, would never, in actual practice, be an absolutely straight line from A to B. It would be in all practical cases an irregularly curved line, similar, for instance, to some of the actual stone curves shown in Fig. 71, p. 193, or it might be either convex like the curve No. 3_2, Fig 73, or concave like No. 3_1. These curves may be redrawn in exactly the same way as curve No. 3, and if the lower end of each is assumed to start at point b where the new base line or bq crosses the parabola, we should have for No. 3_2 the new curve $bg_2h_2j_2$, etc., and for No. 3_1 the curve whose beginning is shown by bh_1j_1, etc. Thus again

it is seen that the stone curves No. 3_2 and No. 3_1 on the original full-size diagram are accurately represented also by the curves $bg_2h_2j_2$, etc., bh_1j_1, etc., drawn to the smaller scale on the same piece of paper.

Thus far only the principles involved in understanding the curves and replotting them have been considered. The result at which we are aiming is the determination of the percentage of each material which will be required in the final mixture of the aggregates. Let us first take for this curve No. 3. The curve of stone No. 3 ends at *B*, which indicates that all of this stone is larger in diameter than 0.75 inches (although about 4% of it, for instance, is smaller than 0.80 inches in diameter). Now following up from *B* on the vertical line which represents 0.75 inches in diameter until we come to the parabola at point *b*, we see that the parabola demands that $\frac{bB}{CB}$ or $\frac{61}{100}$ or 61% of all the stone and sand in the entire mixture of stone and sand shall be smaller than 0.75 inches in diameter, and conversely that $\frac{bC}{CB}$ or $\frac{39}{100}$ or 39% of the mixture shall be larger than 0.75 in diameter. No. 3 stone is the only one of the three lots of stone which is larger in diameter than 0.75 inches, and therefore 39% of this grade of stone should be used in making up the mixture.

These ratios give us a clue to the method of plotting the curves to the smaller scale with the aid of the slide rule, instead of employing the longer method of actually dividing the spaces into 100 equal parts. The principle in each case is exactly the same. By the method of ratios the curve *bkA* would be plotted from the knowledge that $\frac{Cb}{CB} = \frac{Tg}{TG} = \frac{Sh}{SH} =$, etc. The distances *Tg*, *Sh*, etc., may be read directly from the slide rule or from the equation which follows from the preceding, viz., that $Tg = \frac{TG \times Cb}{CB} = \frac{96 \times 39}{100} = 37\%$, and so on.

This actual plotting of the curves may be unnecessary, in fact, it is usually unnecessary for an experienced calculator, as the percentages can be obtained and the general direction of the curve estimated by inspection.*

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*It is evident that neither of the two batches or lots of materials shown by curves No. 3_2 and No. 3_1 are so well adapted to form a parabola as curve No. 3 Curve No. 3_2 would more nearly fit the parabola than it now does if its new curve were plotted slightly lower so that it would cross the parabola at a different point and a smaller percentage of it would be required for the mixture. If it crossed the parabola at *V*, the percentage of it to use could be found by plotting it in this new location and taking for the percentage the vertical distance from *C* to the end of the curve, or what is the same thing, taking the percentage as $\frac{SV}{SH_2} = \frac{33}{65} = 51\%$.

The next curve in order is No. 2. We note that this lot of stone is the only one of the three whose particles lie between 0.25 inches diameter and 0.75 inches, and that therefore all of the stone called for by the parabola between these two sizes must be supplied from No. 2 lot. Following down from the upper end, *C*, of No. 2 to the parabola at *b* and up from the lower end *E* to the parabola at *e* and drawing horizontal line *ex*, we see that the proportion of No. 2 stone which is called for by the parabola is represented by the distance between the lines *rq* and *ex* or by line *re*, and we have the ratio $\frac{re}{DE} = \frac{26}{100} = 26\%$, as the percentage of the weight of the No. 2 material to the total weight of the mixture.

Plotting curve No. 2 in its new location as a part of the mixture we have the dotted line *eb* as representing the No. 2 material after it becomes a part, that is, 26%, of the mixture. The upper end must join the line *bA* because we are now plotting a curve which represents a mixture of the two materials, No. 3 and No. 2, and the mixture must be represented by one single, continuous curve. We may consider *rb* and *ex* as two base lines, divide the vertical distance between them into 100 parts, and then plot the percentages downward from *rb*, equivalent on the small scale to the percentages downward from *DC* to the original No. 2 curve *CE*, as described on page 198, or we may take ratios, as described on page 200, and using the slide rule set *DE* (100) on *De* (65) and on any vertical distance from *DC* to the line *CE*, we may read the distance from *rb* to the resultant curve *eb*. In practice, the line *rb* need not be plotted, but each ratio as it is obtained may be added to the per cent. already found for the No. 3 material to obtain the distance down on the ordinate for the final curve of the mixture, as shown on page 208.

The required percentage of material No. 1 may be obtained by deducting the sum of the percentages of No. 2 plus No. 3 from 100, or by inspection of the parabola and the curve of the portion of the final mixture already plotted, *ebkA*. From the location of the point *e* it is evident that 35% of the total mixture of the material must pass a 0.25-inch sieve. Since No. 1 is the only material whose particles are finer than this, it is evident that this percentage of the total mixture must be entirely formed by No. 1. In other words, the percentage of No. 1 to the total mixture of 100 parts is 35%. To plot the curve *OD* as a part of the mixture, we may divide the distance *eE* into 100 parts, and plot the percentages, or we may take the slide rule and set *Ee* on *DE*, that is, 35 on 100, and read the correspond-

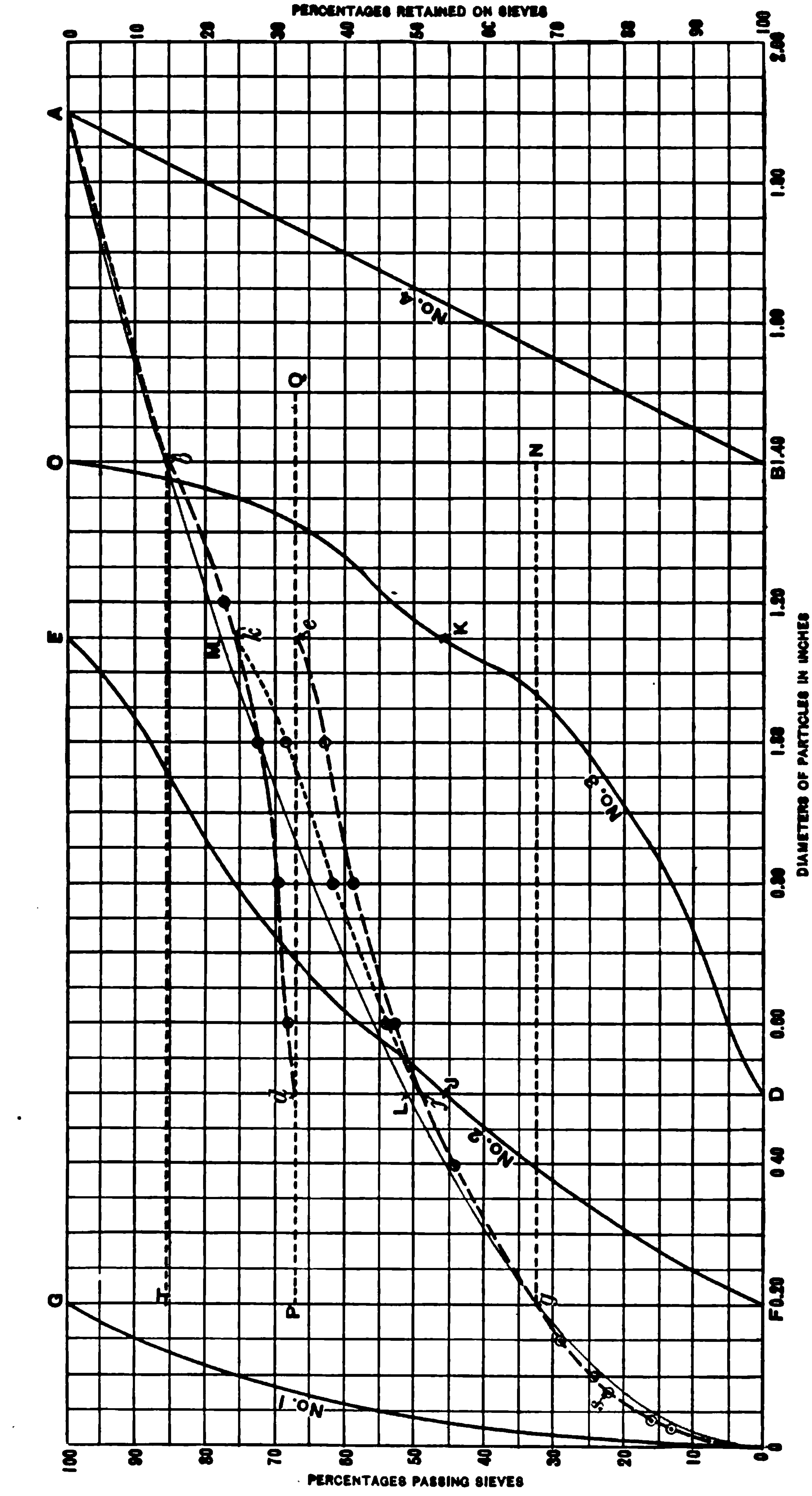

FIG. 74. — Diagram Illustrating Method of Combining Curves which Overlap. (*See pp.* 203 *to* 205.)

ing ratios for the other ordinates. Thus, at ordinate 0.10, $DE : eE = ZW_1 : zW_1$, or $100 : 35 = 71 : zW_1$, hence $zW_1 = 25$.

The final curve of the mixture of materials No. 3, No. 2, and No. 1 in proportions represented by the percentages obtained is represented by the dotted line *AkbezO*.

To illustrate how simply such a diagram as Fig. 73 is solved in practice without really going through the processes described, we may determine the percentage by weight of each material to the weight of the final mixture as follows:

For material No. 3, $\frac{Cb}{CB} = \frac{39}{100} = 39\%$

For material No. 2, $\frac{re}{DE}$ or $\frac{De - 39}{DE} = \frac{26}{100} = 26\%$

For material No. 1, $\frac{Ee}{ED} = \frac{35}{100} = 35\%$

We have thus the percentages of each aggregate material which must be contained in the total mixture of aggregate. The actual proportions of the concrete expressed in parts are obtained in the same manner as is described for example 2 on page 209.

Case II. Curves which overlap. Fig. 74 shows a more complicated combination of materials than Case I. Curves of four materials are drawn.

From the foregoing it is clear that the percentage for material No. 4 is represented by *Cb* or 14%. Since curves No. 2 and No. 3 overlap each other, their values are less easily determined, and we may leave them and first take No. 1. Curve No. 1 is determined and may be plotted in the same way as curve No. 1 in diagram, Fig. 73, p. 196, giving the curve *Osg*, and the percentage $\frac{gF}{GF} = \frac{33}{100} = 33\%$ the percentage by weight of No. 1 in the final mixture.

Having found the per cent. of No. 1 sand to use and also of No. 4 stone, namely, 33% for No. 1 and 14% for No. 4, we have left 53% of the total mixture which must be made up from No. 2 and No. 3 lots.

On curve *FE* the portion from *E* to *J* is overlapped by that part of the *DC* curve extending from *D* to *K*. We note first that about 20% of the material in the parabola (that portion extending from *g* to *L*) must be supplied with stone from the No. 2 lot, while about 10% of the material of the parabola (the portion extending from *b* to *M*) must come from the No. 3, or *DC* curve. There is left then $53\% - (20\% + 10\%)$ = about

23% of the parabola which must be supplied from the overlapping portions of the two curves. Judging from the general appearance of the two curves it would appear that No. 2 curve contained stone more nearly corresponding to the needs of the parabola than *DC*.

For a trial, therefore, we will give a larger proportion to No. 2 than to No. 3 stone, say, 14% of the remaining 23% to No. 2 and 9% to No. 3. No. 2 stone must then furnish 20 + 14 = 34% of the final mixture and No. 3 must furnish 10 + 9 = 19% of the final mixture. Through *g* draw a base line *gN* on which to construct the new curve for *FE*. 34% higher up draw line *PQ* which forms the upper limit for new curve to represent *FE* and the lower limit for new curve to represent *DC*. Then 19% higher up draw line *bT*, which forms the upper base line for new curve to represent *DC*.

Now, by dividing the vertical distance between the lines *gN* and *PQ* into 100 equal parts, — or else by ratios, taking the slide rule and setting *Pg* on *GF* and reading from the ordinates of *FE*, the distances from the base line *gN* to the points which locate the curve *ge*, — we can readily transfer curve *FE* into the new curve indicated by the dotted line *ge* which is assumed to supply 34% of the stone still needed by the parabola, and in the same way by dividing the vertical distance between the lines *PQ* and *Tb* into 100 equal parts, — or else by taking ratios, — the new *db* curve can be laid down.

The curve from *g* to *j* and from *b* to *k* remains as it is.

With a pair of dividers transfer the distance at each ordinate from base line *PQ* up to curve *db* down to curve *ge*, and add it to the curve. These new points will give the dotted curve *jk* as the exact location of the two batches of stone No. 2 and No. 3 combined, 34% of the one being used and 19% of the other.

The resultant curve, *jk*, may be found in another manner after selecting the percentages of the different materials by adding on any ordinate the percentages of each material in the final mixture. For example, on 1.00 diameter, 26% of No. 3 stone passes a 1-inch sieve, but since No. 3 actually occupies only 19% of the mixture, the percentage of No. 3 stone passing the 1-inch sieve in terms of the weight of the total mixture (which is 100%) would be 19% of 26% = 5%. Similarly, the percentage of the portion of the No. 2 stone in the final mixture which passes a 1-inch sieve is 34% of 88% or 30%. All of the No. 1 material (33%) passes the 1-inch sieve, so this too must be added to the others, and we have 5% + 30% + 33% = 68% as the percentage of the final mixture which will pass a 1-inch sieve.

An inspection of this dotted line *jk* resulting from combining these

curves leads us to the conclusion that we should have done rather better to have taken more of No. 2 stone, say, 38% instead of 34%, and 15% of No. 3 instead of 19%, in which case the combined curve would have more nearly corresponded with the parabola. We would have, therefore, as a result of our study the required percentages of material as 14% of No. 4, 15% of No. 3, 38% of No. 2, and 33% of No. 1.

Practical Examples of Proportioning. Having taken up in a very elementary fashion the principles by which curves are drawn and combined, we may take two examples of other combinations of materials liable to be met with in practise.

Example I. — Curves of two materials. Suppose we have for concrete the fine sand of Fig. 72, p. 194, to use with the crushed stone of Fig. 70, p. 192, what proportions of each should be employed and how could the mixture be improved?

DIAMETERS OF PARTICLES IN INCHES

FIG. 75. — Method of Proportioning Two Aggregates. (*See p.* 205.)

Solution. — The curves of the two materials are plotted to the same scale in Fig. 75 as *OF* and *DBLA*, and then the parabola *OCA* drawn by the method previously described.

The parabola indicates that for a theoretical mix of sizes of aggregate up to 1¾ inches, 93% of the mixture should pass a 1½-inch sieve, 76% should pass a 1-inch sieve, 53% a ½-inch sieve and so on.

Where, as in this case, the materials to be mixed are represented by only two curves, no combination of which will make a curve as close to a parabola as is desirable, there is another limiting condition which was

brought out by the experiments, viz., that for the best results the combined curve shall intersect the parabola on the 40% line, at *C*, and that the finer material shall be assumed to include the cement.

In this case, therefore, where the stone and sand curves do not overlap each other, to determine the best proportions of stone and sand, we have merely to take such proportions of each that the resultant curve will pass through the parabola at the point *C* where it crosses the 40% abscissa. This percentage is obtained by taking the ratio $\frac{EC}{EB} = \frac{60}{98} = 61\%$. The percentage by weight of sand plus cement to total aggregate will be 100% — 61% = 39%. The curve of the mixture may now be drawn by replotting the curve *DBLA* in its new location *JCGA* and the curve *OF* in its new location *OJ*, thus making the combined curve *OJCGA*.

Now decide upon the amount of cement to use in the mix to give the required strength of concrete, say, one cement to eight aggregate (the proportion of aggregate being based on measurement before mixing together the sand and stone), which will make the cement one-ninth or 11% of the total materials. Deducting this from the sand plus cement, we have 39% — 11% = 28% sand, and our best proportions for a 1:8 mixture will be 11 parts cement: 28 parts sand: 61 parts stone, which is equivalent to 1: 2.5: 5.5. If the proportions are required by volume and the relative weights of the sand and stone differ from the relative volumes, the proportions should be corrected accordingly.

Plotting the analysis curves of the two materials, as described above, shows immediately how to improve the mix. If, for instance, the crushed stone had been better proportioned so as to contain more particles of 0.5 and 1.0 inch diameter, — see curve *DHA*, — a curve much nearer the parabola could have been constructed. In this case the ratio would have been $\frac{EC}{ER} = \frac{60}{91} = 66\%$ of stone, and the proportions of cement, sand, and stone for a 1: 8 mixture, 11: 23: 66 or 1: 2:6, a stronger and a more impermeable mix. A still better mixture would have resulted with the use of coarser sand having a curve similar to the broken line *OMN*, which with the first material, *DBLA*, would have brought the continuous line of the mixture very much nearer the parabola, by using the ratio $\frac{MC}{MB} = \frac{45}{83} = 54\%$ of curve *DBLA* and 46% of curve *OMN*. This method thus shows not only the best proportions for given materials, but also the defects in the materials and how to remedy them.

The most valuable use of the method of proportioning by mechanical analysis is in cases where the character of the work warrants employing several grades, that is, several sizes, of stone and sand. Such mixtures are being increasingly employed as engineers and contractors more fully appreciate the necessity of so economically proportioning the materials as to produce a mixed aggregate of the greatest possible density, — that is, with the fewest possible voids, — thereby reducing the quantity of cement and at the same time improving the quality of the concrete, in other words, making both a better and a cheaper concrete.

The process of determining the percentages of each material is more complicated than where only two aggregates, sand and stone, are used, but it is not very difficult in practice, especially if one is familiar with the slide rule, and, as illustrated in Example 2, the method is more exact than with two materials, for the reason that the resulting curve can be made to more nearly approach the parabola.

DIAMETERS OF PARTICLES IN INCHES

FIG. 76. — Method of Proportioning a Graded Mixture. (*See p.* 207.)

Example 2. — *Graded Materials.* Given the medium sand, represented by curve in Fig. 72, page 194 and the three sizes of crushed stone represented by the curves in Fig. 71, page 193, find what percentage of each will best combine to make the strongest and most impermeable concrete.

Solution. — Since mechanical analysis of each material has already been made, we will immediately replot the four curves on the same scale in Fig. 76 and draw parabola passing through point O and the point at which curve No. 4 reaches 100%. We see at once that percentage of No. 4 stone required is $\frac{Kk}{KB} = \frac{36}{100} = 36\%$. (To be sure, about 8% of No. 4 is overlapped by No. 3, but this is so slight it need not here be considered.)

Let us determine sand curve No. 1 at 0.10 diameter ordinate, since it can be seen by inspection that the portion *oh* of curve No. 1 very nearly fits the parabola and grains smaller than 0.10 diameter must be supplied wholly from this curve, while the larger grains represented by portion *hG* are found also in No. 2 curve. Accordingly, we have the percentage $\frac{Ff}{Fh} = \frac{20}{88} = 23\%$.

A part of No. 3 curve, that portion extending from *D* to *l*, is overlapped by nearly the whole of No. 2 curve. We can see, however, that No. 3 curve alone must supply 14% of the material in the parabola (that portion extending from *e* to *k*). This leaves 100 — (36 + 23 + 14) = 27% of the mixture to be furnished by the overlapping portions of No. 3 and No. 2 in such ratio as best fits the parabola.

From a study of the two curves, we find by inspection and trial plottings that most of the material required would be better supplied by No. 2 curve, since it contains stone corresponding very well to the needs of that part of the parabola extending from *f* to *e*. Let us consider 23% as the proper amount of the final mixture to be furnished by No. 2 curve, which would leave 14 + 4 = 18% as the total portion which must be supplied by No. 3 curve.

Now, on any of the ordinates, we can locate points through which a curve may be drawn which represents a mixture of the given sand and stone in the proportions just found, for example:

Ordinate.		% Retained.
1.75	40 x 36%	= 14
1.50	57 x 36%	= 20
1.10	92 x 36%	= 26
1.00	(100 x 36%) + (8 x 18%) = 36 + 1	= 37
0.80	36 + (31 x 18%) = 36 + 6	= 42
0.60	36 + (66 x 18%) = 36 + 12	= 48
0.40	36 + (88 x 18%) + (21 x 23%) = 36 + 16 + 5	= 57
0.30	36 + (93 x 18%) + (40 x 23%) = 36 + 17 + 9	= 62
0.15	36 + 18 + (92 x 23%) + (6 x 23%) = 36 + 18 + 21 + 1	= 76
0.05	36 + 18 + 23 + (30 x 23%) = 36 + 18 + 23 + 7	= 84

These percentages are plotted on the diagram as small circles. The same points would have been obtained if we had begun at the left of the diagram and calculated the percentages passing the sieve.

We find that a curve drawn through these points satisfies the parabola sufficiently well to assume that 23% of sand, 23% of finest stone, No. 2, 18% of medium stone, No. 3, and 36% of the largest stone, No. 4, would make the best concrete mixture out of the given materials.

If 1 : 7 concrete is wanted there would be * $\frac{100}{7}$ = 14.3 parts cement, and the proportions would be 14 : 23 : 23 : 18 : 36 or 1 : 1.6 : 1.6 : 1.3 : 2.5 by weight. This would give very nearly an ideal mix, and the resultant concrete would be impermeable and very strong.

VOLUMETRIC SYNTHESIS OR PROPORTIONING BY TRIAL MIXTURES

Having determined the particular sand and stone which are to be used on any piece of work, a simple and accurate way of determining proportions is by actual trial batches of fresh material. For this it is only necessary to have a good scales and a strong and rigid cylinder, say, a piece of 10-inch wrought iron pipe capped at one end. Carefully weigh out and mix together on a piece of sheet steel or other non-absorbent material all the ingredients, having the consistency the same as is intended to be used in the work. Place these in the pipe, carefully tamping all the time, and note the height to which the pipe is filled. Weigh the pipe before filling and after being filled, thus checking weight of material mixed. Throw this material away before it has time to set and clean the pipe. Make up another batch, using the same weights of cement and water and the same total weight of sand and stone, but have the ratio of weights of the sand and stone slightly different from the first. Note, if after placing, the height in the cylinder is less or more than the first, and this will be a guide to further similar mixes, until a proportion is found which gives the least height in the cylinder, and at the same time works well while mixing and looks well in the cylinder, all the stones being covered with mortar. This method, if carefully followed, will give very accurate results, but of course does not indicate what other changes can be made in the physical sizes of the sand and stones so as to get the best available composition as can be done by mechanical analysis.

Mr. A. E. Schutté, in studying the proportions of materials for bituminous macadam pavement for the Warren Brothers Company, has very effectively developed the method of volumetric synthesis with dry materials. His experiments included various classes and sizes of stone, sand, and screenings ranging from 3 inches diameter down to that which passes a No. 200 sieve. He found that the best method for compacting dry materials, such as sand, gravel or broken stone, is to place them in a vessel the shape of a truncated cone, with the largest diameter at the bottom. The cone is filled with the coarsest material and taken by a laborer who compacts it by repeatedly striking the cone against the ground, keeping

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* In this case the finest material does not include the cement.

the measure full by adding new material of the same kind. When it ceases to settle the contents is emptied and mixed with a portion of a finer material, replaced in the measure and compacted as before. By repeated trials the exact size and maximum volume of successive finer materials, which may be added without appreciably increasing the bulk of the coarsest after thoroughly compacting, are determined. Mr. Schutté has found that for different shapes of particles the proportions of each size must be varied, but having determined the required percentages for a certain stone, that is, for a stone from a certain quarry, the proportions of the sizes from day to day need be varied but little.

PROPORTIONING BY VOID DETERMINATION

If the stone or gravel is found to contain, say, 40% voids, as measured by the contained volume of water, the required volume of sand is theoretically 40% of the volume of the stone, and supposing the ratio of cement to sand to be as 1 : 2, the relation of parts of sand to parts of the coarse aggregate would be as 2 : 5, thus making the proportions 1 : 2 : 5. Because of the inaccuracy of this method of procedure, discussed on p. 184, it is necessary in most cases, even although the cement and water will still further increase the bulk, to take a volume of sand, say 5% to 10% in excess of the voids; that is, for gravel with 40% voids to use 45% to 50% of its volume of sand, thus making the proportions 1 : 2 : 4½. If the coarse material is screened broken stone of large size, say 1½ or 2-inch, the volume of sand may be taken equal to the volume of voids instead of in excess of them, because the particles of sand will all be small enough to fit into the voids of the stone without appreciably increasing its bulk. Such stone usually has about 45% to 50% voids, so that we should have proportions 1 : 2 : 4½ or 1 : 2 : 4, the same as for the gravel concrete.

The irregular distribution of the materials by imperfect mixing may usually be disregarded, because the volume of gaged mortar is always in excess of the volume of sand from which it is made.

Care must be exercised in any case to guard against a larger excess of sand than is absolutely necessary, because the voids in a concrete are lessened by using stone in place of sand. Take, for instance, sand having 45% voids and stone having 40% voids. With the sand just filling the voids of the stone it is easily calculated that the resultant mass has 18% voids; but supposing an excess of 10% of sand, there would be 10% of the material having 45% voids, which means there would be 2.5% more voids in the resultant mass.*

*See discussion by the writer in Transactions American Society of Civil Engineers, Vol. XLII, p. 142.

Authorities differ as to whether the stone should be loose or shaken when determining the voids. The writer prefers loose measurement because it corresponds more nearly to the final volume of the concrete, and more sand is always necessary than will just fill the voids of rammed stone, since the sand and cement separate the stones and prevent their lying close together in concrete. In determining, however, the quantity of cement required for the mixture of aggregates the materials should be compacted as described on page 213.

RAFTER'S METHOD OF PROPORTIONING

Mr. George W. Rafter* has called attention to the method of proportioning the mortar as a percentage of the volume of the stone slightly shaken, the relation of cement to sand having been determined by the required strength of concrete.

Quoting from specifications for the Genesee Dam, the concrete is proportioned as follows:

> In forming concrete such a proportion of mortar of the specified composition will be used as may be found necessary by trial to a little more than fill the voids in the aggregate. Tests of the voids will be made from time to time under the direction of the engineer, and instructions given as to the per cent. of mortar of the specified composition to be used. For the information of the contractor, in the way of computing the cost of concrete of the quality herein required, it may be stated that ordinarily the per cent. of mortar will be about 33 per cent. of the measured volume of the aggregate. In case of the use of a certain proportion of gravel in the aggregate, the proportion of mortar may be reduced to somewhat less than 30 per cent.

This method of proportioning is more accurate than the usual procedure, because there is less apt to be an excess of mortar. It does not, however, take account of the fact that with a graded coarse aggregate some of the grains of sand are too large to fit into the voids of the stone, and that therefore the coarse and fine aggregates must be studied together.

FRENCH METHOD OF PROPORTIONING

In France proportions are ordinarily stated in terms of the volume of mortar to the volume of stone, and the mortar is described by the

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* "On the Theory of Concrete," Transactions American Society of Civil Engineers, Vol. XLII., p. 104.

number of kilograms of Portland cement to 1 cubic meter or liter of sand.

PROPORTIONING BY ARBITRARY SELECTION

The arbitrary selection of proportions, with no reference whatever to the voids, if the sand or dust has been screened from the coarse material, for reasons discussed in the preceding paragraphs, gives much better results in practise than might be expected. To be sure, the voids in a screened broken stone, loosely measured, may be 50%, while a gravel with the sand screened out may have only 40% voids, but, nevertheless, the proportion of sand required in the two cases may vary but little, because, as stated above, the voids in the gravel are of smaller dimensions and a larger excess of sand is necessary.

The percentage of volume of sand to ordinary gravel or broken stone from which the finest material has been screened may be taken between the limits of 40% and 60% with an average, which is suitable under many conditions, of 50%. This ratio corresponds to proportions 1 : 2 : 4, 1 : 2½ : 5, 1 : 3 : 6, and 1 : 4 : 8, which are given by the authors in Chapter II as standard mixtures for the use of those who are inexperienced in concrete work. In some cases, especially where the coarse material contains a good many small particles, as does crusher-run broken stone, the proportion of sand may be made slightly less than half the volume of stone.

Unscreened gravel is often used alone for the aggregate with good results, but more uniform conditions can be maintained, and therefore leaner proportions employed, by screening out and remixing the sand.

Proportions adopted by various authorities and tabulated on page 214, may serve as a guide to arbitrary selection.

SCREENED VS. UNSCREENED GRAVEL OR BROKEN STONE

The advisability of using screened or unscreened gravel or broken stone is a matter of economy. The writer is in favor of separating the aggregate into as many parts as is consistent with economy for the work in hand. Even on small jobs he believes it preferable to screen out the sand or the dust and remix it in specified proportions, because of the variation in the quantity of sand in different parts of the same gravel bank, and of the separation of the coarse stone from the dust as it rolls down the pile. Good concrete is doubtless sometimes made from unscreened gravel and crusher-run stone to which a little sand has been added, but the cost of screening except for the very smallest jobs is much less than the cost of

the additional cement necessary to secure the same strength or impermeability with the unscreened material.

DETERMINATION OF THE PROPORTION OF CEMENT

The discussion thus far has related chiefly to the proportioning of the aggregates, that is, of the sand and stone. One of the prime objects in arranging the volumes of these inert materials to produce a dense mixture is to reduce the quantity of cement. The selection of the proportion of cement in concrete is to a certain degree a matter of judgment, because the qualities of strength, denseness, impermeability, and practical working in the field must be considered with greater or less emphasis upon one or another according to the character of the work.

To determine the minimum quantity of cement which will produce a concrete practically free from air voids, the aggregates are mixed in the correct proportions as described in the preceding pages, compacted by ramming or hard shaking, and their voids determined by weighing and correcting for variations in specific gravity.* The sand should be in the natural state of moisture found in the interior of the bank, not because this is the condition in which it will be mixed in the concrete, but because it may be assumed in the natural state to contain a quantity of moisture varying with its fineness. If gravel is used it may be taken in the same way, while coarse broken stone should be dry, and dry broken stone screenings may be mixed with about 4% of water by weight. Correction must be made for this moisture after weighing the mixed material, so that the voids calculated will be simply air voids.

In determining the quantity of cement to fill these air voids it may be assumed without appreciable error that 100 lb. of cement will make 1.0 cu. ft. of neat paste. This is a larger volume than would result with ordinary plastic paste, but makes a slight allowance for the additional moisture required for the sand and stone. To the quantity of cement thus determined 10% may be added, *i.e.*, 10% of the cement, not of the total mixture) to provide for imperfect mixing.

PROPORTIONS OF CONCRETE IN PRACTICE

The proportion of cement to the aggregate depends upon the nature of the construction and the required degree of strength or water-tightness as well as upon the character of the inert materials. Strength and impermeability are discussed in Chapters XIII and XX respectively, but the

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*See p. 163.

Proportions in Actual Structures.

Compiled by TAYLOR AND THOMPSON.

Structure.	Nominal Proportions.	Portland Cement bbl. of 380 lbs.	Loose Sand cu. ft.	Loose Stone cu. ft.	Volumes based on nominal or actual measurement.	Authority.	Reference.
New Brooklyn Bridge Piers . . .		1	8.5	19.5	nominal	Asst. Engineer	
Boston El. Ry. Column Foundations	1:2½:5*	1	9.5	19.1	nominal	G. A. Kimball	Jour. A. E. S. June '03, p. 353
N. Y. C. & H. R. R. R.							
Footings	1:4:7½†	1	13.9	26.2	actual	W. J. Wilgus	Assn. of Ry. Supts. 1900, p. 207
Abutments	1:3:6†	1	12 2	23.7	actual		
Facing Old Masonry	1:2:4	1	7.0	14.0	actual		
Coping and Bridge Seats . . .	1:1:2	1	3.5	7.1	actual		
C. M. & S. P. Ry.							
Piers and Abutments	1:2:5	1	7.8	21.4	actual	W. A. Rogers	Assn. of Ry. Supts. 1900, p. 228
Culverts and Foundations . .	1:3:7½	1	10.5	28.5	actual		
Or. R. R. & Nav. Co.							
Abutments, Piers and Culverts .	1:3:5	1	11.0	18.3	nominal	W. H. Kennedy	Assn of Ry. Supts. 1900, p. 182
Foundations and Light Buildings	1:3½:6	1	12.8	22.0	nominal		
	1:4:7	1	14.7	25.7	nominal		
C. & E. I. R. R.	1:2:5	1	9.3	26.7	actual	A. S. Markley	Assn. of Ry. Supts. 1900, p. 245
Northern Pacific Ry.							
Foundations	1:3:5‡	1	11.2	20.2	actual	E. H. McHenry	Assn. of Ry. Supts. 1900, p. 235
Abutments and Piers	1:3:5	1	11.2	20.2	actual		
C., B. & Q. R. R.	1:3:6	1	12.5	22.5	actual	Fred Eilers	Assn. of Ry. Supts. 1900, p. 231
Mexican Central Ry.	1:3:6	1	13.5	27.0	nominal	Lewis Kingman	Assn. of Ry. Supts. 1900, p. 212
N. Y. Subway						N.Y.R.T. Com.	Spec. 1900, p. 83
Roofs and Sidewalls not over 18 in. thick . . .	1:2:4	1	7.2	14.4	nominal		
Sidewalls or Tunnel Arches . .	1:2½:5	1	9.0	18.0	nominal		
Wet Foundations not over 24 in. thick . . .	1:2:4	1	7 2	14.4	nominal		
Wet Foundations exceeding 24 in. thick . .	1:2½:5	1	9 0	18.0	nominal		
Boston Subway	1:2½:4	1	8.3	13.2	nominal	H. A. Carson	
Harvard University Stadium . . .	1:3:6						
Maine Fortifications							
Leveling for Foundations . . .	1:5:10	1	18.2	36.5	nominal	S. W. Roessler	Report Chief of Engrs. U. S. A. 1901, p. 911
Walls and Masses not exposed to fire	1:4:8	1	14.6	29.2			
Walls and Masses exposed to fire	1:3:6	1	11.0	22.0	nominal		
Masses for greater imperviousness	1:3:5	1	11.0	18.3	nominal		
Little Falls						W. B. Fuller	
Mass Concrete	1:3:7	1	11.4	26 6	nominal		
Tanks, Buildings, etc., ...	1:2:4	1	7.6	15.2			
Duluth Ship Canal Piers		1	11.8	23.8	nominal	C. Coleman	Cement, Sept., '00, p. 144
Boonton, N. J., Dam	1:2¾:6¼§	1	10.5	23.8		W. B. Fuller	
Genesee Dam	33% mortar	1	11.4	36.8	nominal	Geo. W. Rafter	
Buffalo Breakwater		1	5	30‖	nominal	Emile Low	Trans. A. S. C. E. Vol. LII, p. 102
Pennsylvania Tunnel	1:2½:5	1	9.6±¶	10.3±¶	nominal	Specifications	Eng. News, Oct. 15, '03, p. 337
East Boston Tunnel	1:2½:4	1	7.7	12.4	nominal	H. A. Carson	Specifications, 1900

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* Mixture varied with loading from 1:1:3 to 1:3:6.

† 25% of the mass is rubble.

‡ Boulders added.

§ 55% of the mass is rubble.

‖ 15 cu. ft. gravel and 15 cu. ft. broken stone Actual volumes of aggregates, 25% higher.

¶ The specifications give proportions in volumes shaken, hence 10% has been added to convert them to loose measurement.

table which follows, compiled by the authors, giving the proportions adopted upon important structures, may in some cases be useful as an arbitrary guide. Actual measurement, that is, measurement of proportions as actually used, almost invariably shows leaner mixtures than the nominal proportions called for. This is largely due to the heaping of the measuring boxes in practise.

In general, as both strength and imperviousness increase with the proportion of cement to aggregate, relatively rich mixtures are necessary for loaded columns and beams in building construction, for thin walls subjected to water pressure, and for foundations laid under water.

CHAPTER XII

TABLES OF QUANTITIES OF MATERIALS FOR CONCRETE AND MORTAR

This chapter presents tables, curves, and formulas (pp. 221 to 235), by which the volumes of materials required for a known volume of concrete may be estimated, and emphasizes the importance of distinctly stating the proportions (p. 217).

The volume of concrete, even when made from materials in the same proportions, varies largely with the character of the materials and the methods of placing it. A mixed aggregate like gravel contains fewer voids and with the same proportions by volume of the same cement and sand produces a larger quantity of concrete than a screened broken stone. The fineness of the sand also largely affects the volume of the concrete and mortar, a fine sand requiring more water, and therefore producing a larger volume of mortar than coarse sand in the same proportions by volume. If the sand is dry, a slightly larger bulk of mortar is produced than with the same sand when containing a larger percentage of moisture, because the latter is less compact (see p. 176). Some cements require more water in gaging than others, and produce a larger amount of paste, which increases the volume of the concrete or mortar. The method of mixing and placing the concrete also affects the resulting volume, since an imperfectly mixed or poorly compacted mass contains voids which increase the volume. An excess of water in mixing affects the resulting volume of the set concrete or mortar to a slight extent, although most of the surplus water is expelled during setting.

It is possible to provide for all these variations, except those relating to improper mixing and placing, in rational formulas from which the resulting volumes may be accurately estimated if the characteristics of all the materials are known. For most practical purposes, however, average values, such as are presented in the tables and curves, are sufficiently accurate for estimating quantities. These average values are based upon a large number of tests in the United States, France, and Germany.

The theory of a concrete mixture is discussed, and formulas for volumes and quantities are given on pages 220 to 227 preceding the tables.

EXPRESSING THE PROPORTIONS

In framing concrete specifications, the proportions of the constituents should be stated so distinctly that there can be no misunderstanding between the engineer and the contractor as to the quantities which will be required for the work. The quantity of cement should invariably be regulated by its weight; if the proportions are stated by volume a definite weight or number of packages of cement must be assumed to the unit volume. For reasons discussed in Chapter XI, it is also more accurate and scientific to measure the aggregates by weight than by volume, and since with a properly constructed plant using materials of several sizes, the cost need be no more than volume measure, the authors believe this will eventually become common practice in the case of important construction.

With our present system of weights and measures, it is advisable either to specify the number of cubic feet (or pounds) of sand and gravel, stone, or mixed material to a definite weight of cement, or else to stipulate a definite weight of cement to a cubic yard of concrete tamped in place, with an aggregate of clearly described material proportioned as the engineer may direct.

In stating the proportions for both mortar and concrete, it is now customary in the United States to separate the materials by colons, the first figure always representing the cement, followed by the aggregates in the order of the size of their grains. For example, 1: 3: 6 means 1 part cement (the unit of measurement should be stated), 3 parts sand, and 6 parts coarse material; or 1: 8 means 1 part cement (of defined weight) to 8 parts of graded aggregate. Mortar in proportion 1: 2 signifies one part cement to two parts sand by either weight or volume as specified.

In France, proportions are stated as one or more volumes of mortar to a definite number of volumes of stone, — "un volume de mortier pour deux volumes de cailloux."

Unit for Proportioning. If the proportions must be stated in parts, it is recommended that the weight of cement be assumed as 100 lb. per cubic foot, and the corresponding volume of a barrel as 3.8 cu. ft. By this system of units, proportions 1: 3: 6 would represent 100 lb. cement to 3 cu. ft. of sand to 6 cu. ft. of gravel or stone; or, 1 bbl. cement (*i.e.*, 4 bags or 376 lb.) to 11.4 cu. ft. sand to 22.8 cu. ft. gravel or stone.

The authors offer these recommendations after correspondence or personal interview with some fifty authorities* (members of the American

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*See Preface.

Society of Civil Engineers) on concrete construction, representing all sections of the United States.

With reference to the unit which should be selected for the volume of a cement barrel (corresponding to 376 lb. Portland cement) the opinions were varied, but nearly every authority advocated specifying a definite weight of cement instead of measuring it loosely by volume. The units which met with the most favor were 3.5, 3.6, 3.8 and 4.0 cu. ft. The advocates of the first two values based their figures upon the measured volume of a cement barrel, while those selecting the last two did so on the presumption that the unit is an arbitrary one in any case, and 100 lb. per cubic foot, or 95 lb. per cubic foot (the latter equivalent to 1 cu. ft. to the bag), is convenient for calculation. An approximate average of all the figures suggested was 3.8 cu. ft. to the barrel, corresponding to 100 lb. per cubic foot, the advocates of this value being, among others, Messrs. Charles E. Fowler, William B. Fuller, Peter C. Hains, Allen Hazen, Rudolph Hering, George A. Kimball, Leonard Metcalf, J. Waldo Smith, and J. H. Wallace. Accordingly, in cases where it is advisable to specify the proportions by parts, the authors have adopted this unit as their standard.

When stating the proportions by volume, too much stress cannot be laid upon the necessity for the adoption of a standard unit, such as a barrel of 3.8 cu. ft. or the equivalent assumption that a cubic foot of cement weighs 100 lb., and upon distinctly specifying this standard, as otherwise an unscrupulous contractor may adopt for his unit the volume of cement very loosely measured, and thus produce too lean a concrete. Moreover, without a standard there is no means of comparing the concrete in different structures or the results of different experiments. It is even inaccurate to state that proportions shall be based on packed or on loose measurement of cement, for either of these terms is very elastic. The authors have personally known engineers to place the volume of a barrel of packed cement all the way from 3.1 to 3.8 cu. ft., corresponding to a variation in weight of from 123 to 100 lb. per cubic foot, while loose measurement, on the other hand, is variously fixed at from 3.8 to 4.5* cu. ft. to the barrel, or 100 to 84½ lb. per cubic foot. The extreme actual variation is therefore from 3.1 to 4.5 cu. ft. per barrel, or 123 to 84½ lb. per cubic foot. Proportions 1:3:6 in the first case would require 1 bbl. or 376 lb. cement to 9.3 cu. ft. of sand and 18.6 cu. ft. of gravel; in the last case, proportions 1:3:6 would stand for 1 bbl. or 376 lb. cement to 13.5 cu. ft. of sand and 27 cu. ft.

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*This value is given by one engineer in Proceedings Association of Railway Superintendents of Bridges and Buildings, 1900, p. 212.

of gravel. In other words, concrete mixed 1 : 3 : 6 by one man may be called 1 : 4½ : 8½ by another.*

It may be contended that this variation is of little moment provided the unit is distinctly stated. The fact is, however, that it is customary in discussing a piece of work to give the proportions of materials without stating the unit selected, and many records giving tests of strength of concrete do not even specify the units used in proportioning the ingredients. It is especially confusing also, to a contractor who is not very careful in reading specifications, to find that, say, 25% or 30% more cement than he had figured is required to a cubic yard of concrete. When considering this question, the authors were surprised to find that the sidewalk and paving specifications of fifteen of the largest cities in the United States failed to state the proportions by definite weight or volume, but gave the quantities simply in "parts," a few of them adding that the parts shall be "by measure" or "by exact measure."

Tests of Capacity of Portland Cement Barrels and Weight of Contents.

(Tabulated by the authors from measurements of Boston Transit Commission, 1896, Howard A. Carson, Chief Engineer.) (*See p. 219.*)

No. of barrels tested results averaged	Brand	Height between heads	Average diameter of barrel	Average horizontal area	Capacity of barrel between heads	Depression of cement below head	Volume of depression	Volume of cement per barrel			Net weight of cement per barrel		Weight per cubic foot				Weight of barrel
								Packed	Loose	Shaken*	Before dumping	After dumping	Packed	Loose	Shaken	Sifted	
		ft.	ft.	sq.ft.	cu.ft.	ft.	cu.ft.	cu. ft.	cu. ft.	cu. ft.	lb.	lb.	lb.	lb.	lb.	lb.	lb.
5	A	2.12	1.437	1.622	3.446	0.17	0.235	3.21	3.75	3.432	377.4	376.9	117.5	100.5	109.4	90.6	21.1
6	B	2.19	1.430	1.605	3.495	0.12	0.171	3.35	4.17		381.0		113.8	91.4			29.0
3	C	2.07	1.412	1.571	3.249	0.07	0.096	3.15	4.05		387.0		112.8	94.2			22.7
5	D	2.01	1.407	1.554	3.123	0.07	0.093	3.03	3.99	3.522	373.2	371.4	123.2	93.2	105.5		25.6
6	E	2.08	1.403	1.546	3.219	0.04	0.059	3.16	4.19		374.2		118.4	89.2			24.3
1	F	2.13	1.38	1.496	3.186	0.03	0.039	3.15	4.27	3.695	378.0	378.0	120.1	88.5	102.3		22.0
5	G	2.01	1.46	1.662	3.327	0.10	0.148	3.21	4.06	3.598	370.7	370.2	115.7	91.4	102.9	80.3	23.3
Final Averages		2.09	1.42	1.579	3.292	0.09	0.120	3.18	4.07	3.562†	377.4	374.1†	118.8	92.6	105.1†	85.4†	24.0

NOTE.—A and B are American Cements; C. D E and F are German Cements; G is a Danish Cement; Paper weighs about 1 lb.

*Box rocked over bar.

†Partial averages, to be compared only with like brands.

Weight of Cement. Experiments by Mr. Howard A. Carson, for Boston Transit Commission, upon 31 barrels of Portland cement of

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*For further data, see letter of Sanford E. Thompson to *Engineering News*, Nov. 12, 1903, p. 434.

American and foreign brands, furnish an interesting illustration of the difference in weight of the same cement in different stages of compactness. The results,* a summary of which is presented in the table on page 219, show a variation from 86 to 118 lb. in the average weights of the same cement, according as it was weighed, sifted, or packed in a barrel, while the actual weight of one brand, the average of 5 barrels, was as high as 123 lb. per cubic foot as it came from Germany packed in a barrel.

From the experiments just described, the ratios of volume and weight of the same cements in different degrees of compactness are calculated by the authors as follows:

Ratio of volume of packed cement to capacity of barrel between heads	0.97
Ratio of volume packed to volume loose	0.78
Ratio of volume packed to volume shaken	0.88
Ratio of volume loose to volume shaken	1.13
Ratio of weight packed to weight loose	1.28
Ratio of weight packed to weight shaken	1.13
Ratio of weight packed to weight sifted	1.37

From the table it is evident that the selection of the volume of a barrel is arbitrary. The adopted volume of 3.8 cu. ft. is convenient for calculation because it assumes a cubic foot of cement to weigh approximately 100 lb.

THEORY OF A CONCRETE MIXTURE

The discussion and the formulas which follow relate to plastic mortars and plastic or medium concrete. While a small amount of water in mixing may result, with heavy ramming, in a concrete or mortar of less than average volume, in practice the volume is more apt to be increased by lack of water because of the less perfect mixture and the visible voids. The volume of set concrete or mortar produced by a very wet mixture is approximately the same as that of a plastic mixture, because nearly all of the surplus water is thrown to the surface and expelled by the settling of the solid materials. This the authors have repeatedly proved by experiment.

The frequently repeated assertion that a very wet mixture contains visible air voids because of the drying out of the water is incorrect. This may be proved by carefully pouring neat cement grout into a rectangular mold, one of whose sides is formed by a piece of glass. The surplus water is expelled, and the specimen after setting is dense and glassy with no visible voids. The large visible voids which sometimes occur in very wet

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*Tabulated by Sanford E. Thompson in *Engineering News*, Oct. 4, 1900, p. 229.

concrete, similar in appearance to visible voids in dry concrete, are due to the grout running away from the stones, or to too violent agitation in placing.

The volume of fresh concrete or mortar produced by any mixture of cement and aggregate or aggregates is equal to the sum of the volumes of the *separate particles* of the cement, the sand, and the other dry materials, the water contained in the aggregate and added in mixing, and the small volume of air entrained between the particles. The volume of set mortar or concrete is not appreciably different from its compacted volume when fresh or green, except in very wet mixtures, which expel a portion of the water. The volumes of the *particles* of dry materials are termed *absolute volumes*, and it is important to note the distinction between the absolute volumes and the apparent volumes determined by measuring the materials. Absolute volumes are discussed on pages 135 to 139.

The fact that water actually occupies space in a mass of fresh concrete or mortar has been entirely ignored by many writers on the subject of concrete mixtures. As stated on page 216, the fineness of the sand and the moisture contained in it affect the volume of the resulting concrete or mortar. Mr. Feret has proved by experiments (cited on page 179) that fine sands require more water for gaging than coarse. This extra volume of water produces a mortar of less density and consequently less strength; even stones such as are found in gravel or coarse broken stone require a very small percentage of water.

FORMULAS FOR QUANTITIES OF MATERIALS AND VOLUMES

A concrete is therefore made up of solid grains of cement plus water required for the cement, plus solid grains of sand plus water required for the sand, plus solid stone particles plus water required for the stone, plus air voids. The last term, the *air voids*, represents the voids entrained by the sand, which may be considered as a function or percentage of the sand, and the voids due to imperfect mixing of the concrete materials, which may be considered a function or percentage of the stone. Accordingly the volume of a concrete mixture may be expressed as a rational formula, which is applicable to all concrete and mortar mixtures in which the voids of the coarse stone are filled with mortar. The formula (1) which follows is presented to illustrate the theory, but because of the variation in the coefficient with different sands and different proportions, formula (2), page 222, and formulas (3) to (8), which are based on average conditions, are suggested for practical use as sufficiently accurate for most purposes.

Let

c = absolute volume* of cement.

s = absolute volume* of sand.

g = absolute volume* of stone.

m = ratio of the absolute volume of the water plus air voids of the cement, to the absolute volume of cement.

n = ratio of the absolute volume of the water coating the grains of sand plus the air entrained in gaging it, to the absolute volume of sand.

p = ratio of the absolute volume of the water coating the stone particles plus the air voids due to imperfect mixing, to the absolute volume of stone.

W = volume of concrete produced.

In other words, these ratios, m, n, and p, represent the sum of the volumes occupied by the water required for the material in mixing plus the air, in terms of the respective volumes of cement, sand, and stone.

Then

$$W = c + mc + s + ns + g + pg$$

or

$$W = (1 + m)\,c + (1 + n)\,s + (1 + p)\,g \qquad (1)$$

The coefficient n is really composed of two variables, one depending upon the coarseness of the sand, and the other upon the ratio of cement to sand, since a lean mortar contains more air voids. It is possible to express this coefficient as a more complex term with this ratio as a factor, but by what appears to be a peculiar coincidence, experiments show that for ordinary bank sand the variation in voids caused by different proportions may be provided for by taking the cement and sand together; in other words, for different proportions of the same cement and sand, the sum of the water and the air voids in the mortar is approximately a constant. Where there is no sand, or where the stone and sand are mixed, formula (1) must be employed.

The more practical formula may be expressed as follows, employing similar notation to that given above, and letting

r = ratio of the absolute volume of the water plus the air entrained in gaging, to the absolute volume of cement plus sand,

then

$$W_1 = c + s + r\,(c + s) + g + pg$$

or

$$W_1 = (1 + r)\,(c + s) + (1 + p)g \qquad (2)$$

---

*Absolute volumes are defined on p. 135.

Substituting average values for r and p, which the authors have selected by analyzing the results of a number of exact records in the United States and Europe of the volumes of concrete and mortar, the formula becomes

$$W_1 = 1.34\,(c + s) + 1.08\,g \tag{3}$$

The comparison of this formula with actual experiments is shown on page 227. The formula may be readily reduced to practical working form if the characteristics of the cement, sand, and stone are known. The cement may be expressed in pounds by substituting for the absolute volume, c, the number of pounds of cement divided by its specific gravity (which may be taken as 3.1) times the weight of a cubic foot of water (62.3 lb.). It may also be expressed in barrels by substituting for the absolute volume, c, the number of barrels, B, multiplied by the net weight per barrel, 376 pounds, and divided, as above, by the specific gravity times the weight of a cubic foot of water [see formula (4)]. The terms relating to sand and stone may be expressed in pounds in a way similar to that just shown for cement, or they may be expressed in measured volume by substituting for the absolute volume, s or g, the measured volume, S or C, multiplied by the proportion of solid material contained in it. Expressing this algebraically, if

Q = quantity of concrete made with B barrels cement,
Q_1 = quantity of concrete made with one barrel cement,
B = number barrels cement,
B_1 = number barrels cement per cubic yard of concrete,
S = volume of loose sand in cubic feet,
S_1 = volume of loose sand in cubic yards per cubic yard of concrete,
G = volume of broken stone or gravel or cinders in cubic feet,
v = absolute voids in sand determined by weight method (p. 166),
v' = absolute voids in stone determined by weight method (p. 167),

then from formula (3), since $c = B\dfrac{376}{3.1 \times 62.3}$

$$Q = \frac{1.34 \times 376}{62.3 \times 3.1} B + 1.34\,(1 - v)\,S + 1.08\,(1 - v')\,G$$

$$Q = 2.61\,B + 1.34\,(1 - v)\,S + 1.08\,(1 - v')\,G \tag{4}$$

The volume of concrete in cubic feet made by one barrel of cement, assuming that a cubic foot of average loose, moist sand contains 89 pounds of dry sand, and that its specific gravity dry is 2.65, is,

$$Q_1 = 2.61 + 0.723\,S + 1.08\,(1 - v')\,G \tag{5}$$

This formula is applicable to average concrete made with Portland cement of good quality, coarse bank sand measured loose and containing ordinary moisture, and any broken stone or gravel of known voids. Formula (5) has been used in compiling tables on pages 233 to 235, except in the first twelve proportions, which contain no sand.

If the volume of concrete made from a barrel of cement plus the sand and other aggregate which accompanies it is known, the number of barrels of cement per cubic yard is readily calculated. In formula (5), Q_1 represents the number of cubic feet of concrete made with one barrel cement, hence the number of barrels cement per cubic yard of concrete is 27 divided by Q_1

$$B_1 = \frac{27}{Q_1} \tag{6}$$

Assuming a cubic foot of average sand to contain 89 pounds of dry sand produces the formula employed in calculating tables on pages 230 to 232, and substituting in formula (6) the value of Q_1 from formula (5),

$$B_1 = \frac{27}{2.61 + 0.723\,S + 1.08\,(1 - v')\,G} \tag{7}$$

The formulas may be expressed in parts by volume (such as 1:2:4) by multiplying the coefficient of S and G by the assumed volume of a barrel, say by 3.8.

Knowing the number of barrels of cement, B_1, per cubic yard of concrete, the number of cubic yards of sand per cubic yard of concrete, S_1, is evidently

$$S_1 = \frac{B_1 \times \text{quantity sand in cubic feet per barrel of cement}}{27} \tag{8}$$

The quantity of stone is similarly obtained.

If two or more coarse materials, such as broken stone and gravel, are used, they must be mixed in the selected proportions, before weighing, to determine their voids.

In mortars of extremely fine sands the density $(c + s)$ is apt to be about 0.60 (see Feret's table, sand C, p. 136) and the coefficient of first term of formula (3) becomes $\frac{1.00}{0.60} = 1.67$ instead of 1.34. In plastic mortars of standard Ottawa sand the density $(c + s)$, by tests of the authors, averages about 0.71, hence the coefficient becomes $\frac{1.00}{0.71} = 1.41$ instead of 1.34. Substituting these values, or any others which may be obtained by

experiment, in formula (2), the working formulas which follow it may be readily deduced. It is evident from the variation in the coefficient with different sands, that the variation in volume of mortar and concrete obtained by different experimenters is due chiefly to the difference in the materials employed.

The coefficient of $(c + s)$ is also affected, though to a less degree, by the character of the cement, some cements requiring more water than others and therefore producing a greater bulk of paste for a given weight of cement.

In concrete mixtures of cement and coarse stone, with no sand or screenings, formulas (2) to (8) are inapplicable because apparently the air voids do not increase with the leanness of the mixture until the point is reached at which the paste fails to fill the voids in the stone. It is therefore necessary to go back to formula (1), page 222. Since s is zero, the formula becomes

$$W_2 = (1 + m)\,c + (1 + p)\,g \qquad (9)$$

An average value of $(1 + m)$ for a first-class American Portland cement has been found by experiment to be 1.65. It varies with the quantity of water required to gage the cement to such a consistency that the voids will be filled, but no free water will exist upon the surface. The selected value, assuming 1% voids in the paste, corresponds to 20% of water by weight. The value of $(1 - p)$ is usually 1.04 to 1.08. An average formula for a concrete of cement and coarse stone may thus be taken as

$$W_2 = 1.65c + 1.08g \qquad (10)$$

which is readily reduced to practical forms by the method adopted in evolving formulas (4) to (8) from formula (3).

If the stone is a mixture of sand and gravel, or broken stone and screenings, the coefficient of g must be increased and a figure selected whose value depends upon the relative proportion of fine and coarse material.

TABLES AND CURVES OF QUANTITIES OF MATERIALS AND VOLUMES

Tables on pages 229 to 235 are calculated from formulas (5), (6), (8), and (9). These formulas are used not merely because of their theoretical worth, but because, as stated on pages 216 and 227, the results from them agree with actual experiment.

The values are average values of sufficient exactness for practical use, although, as suggested on pages 222 and 224, variations in the

quality of the materials largely affect the resulting volumes, especially of the mortar.

The tables on pages 231 and 234 are recommended for general use in determining the quantities of materials for concrete, or the volume of concrete made with known materials, and where the percentage of voids in the coarse aggregate is unknown the 45% columns should be adopted. The curves on page 228 are also in convenient form for practical use.

All except the first item in the table on page 229 and the first 12 items in tables on pages 230 to 235 are calculated from formulas (5), (6), and (8), page 223, with the assumption there outlined. The broken stone in the first twelve items in the concrete tables, pages 230 to 235, except where the voids are 40% or over, is assumed to contain fine material, and the coefficient selected for *g*, formula (9), varies from 1.08 for 50%, 45%, and 40% voids to 1.14 for 20% voids.

Use of Curves. The use of the curves on page 228 is best illustrated by the following examples:

Example 1. — Find quantities of materials required for 1 000 cubic yards 1:2½:5 concrete.

Solution. — Intersection of dotted horizontal line corresponding to 2½ barrels sand with dotted vertical line corresponding to 5 barrels stone falls on diagonal curve 1.30; hence, 1.30 barrels cement are required per cubic yard, or 1 300 barrels cement for 1 000 cubic yards concrete. From Note 4 of diagram 1 300 × 0.141 × 2½ = 460 cubic yards sand will be required, and 1 300 × 0.141 × 5 = 920 cubic yards stone required.

Example 2. — Find number of barrels cement required for 1 000 cubic yards concrete in proportions one barrel cement to 9 cubic feet sand to 18 cubic feet stone.

Solution. — Intersection of full cross section horizontal line corresponding to 9 cubic feet sand with vertical line for 18 cubic feet stone gives 1.37 barrels cement per cubic yard or 1 370 barrels for 1 000 cubic yards concrete.

Example 3. — Find volume of concrete of Example 1 made from one barrel of cement.

Solution. — By Note 5 of diagram volume of concrete per barrel cement is 27 divided by the quantity of cement per cubic yard of concrete, or $\frac{27}{1.30} = 20.8$ cubic feet.

Comparison of Table Values with Actual Experiments. Comparatively few experimenters have recorded complete data with reference to the materials entering into their specimens of concrete and mortar. The most comprehensive records of this nature that have come to the knowledge of the authors are those by Mr. William B. Fuller,* which are tabulated in full on page 258, his proportions ranging from 1:0 to 1:6:10. The actual volumes obtained by him, having been found to agree closely with other carefully made experiments, are used in the determination of the constants employed in the above formulas and in compiling the tables and curves on pages 228 to 235. Volumes calculated from the formulas employing these constants agree with Mr. Fuller's tests with an average variation of 0.2 of 1% and a maximum variation of 6%.

Other records which have been compared with results calculated by our formulas, and with which they usually agree within less than 5% after making allowance for different materials and units, are those by Messrs. George W. Rafter,† Edwin Thacher,‡ J. E. Howard,§ E. Candlot,‖ and E. S. Wheeler.¶

Experiments by Mr. Edwin Thacher show the rammed volume of dry facing mortar (that is mortar mixed with a small proportion of water) to be about 12% less than the volume of slush mortar made from the same materials, and the quantity of cement per cubic yard to be correspondingly greater for the dry mortar.

The volume of mortar or concrete is affected by the character of the cement as well as by the sand and method of mixing, since some cements require more water and will make more paste to a unit weight of cement than others even of the same class. In one series of experiments, for example, 85 pounds of a certain first-class American Portland cement were required to make one cubic foot of paste, while for another standard American Portland cement of a different brand 107 pounds were required. Average values for wet or plastic mortars are given in the table on page 229.

*See page 261.

†Transactions American Society of Civil Engineers, Vol. XLII, p. 104.

‡Johnson's "The Materials of Construction," 1903, p. 610a.

§Tests of Metals, U. S. A., 1899, p. 786.

‖Ciments et Chaux Hydrauliques, 1898, p. 446.

¶Report Chief of Engineers, U. S. A., 1895, pp. 2922 to 2931.

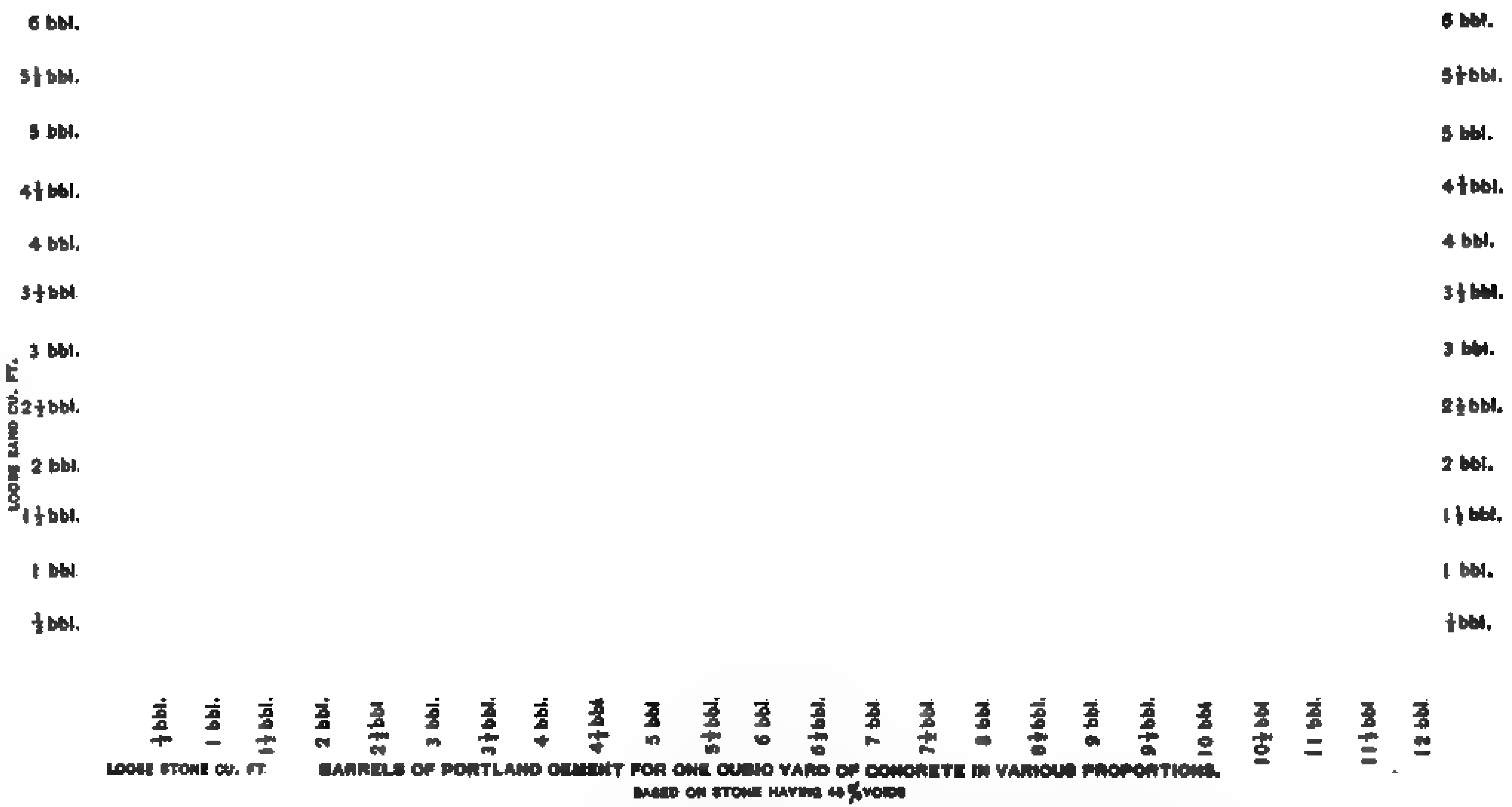

FIG. 77. (See important rules below, also examples on page 226, and formula (7) on page 224.)

1. ***Dotted cross section lines*** represent barrels or parts of sand and stone to one barrel (4 bags) Portland cement weighing 376 lb. and measuring (assumed) 3.8 cu. ft.

2. ***Full cross section lines*** represent cubic feet of sand and stone to one barrel (4 bags) Portland cement.

3. ***Diagonal lines*** represent number of barrels of Portland cement in one cubic yard of compacted concrete.

4. ***To find number of cubic yards of sand or stone*** per cubic yard of concrete, multiply number of barrels cement, as above, by 0.141 times the number of parts of sand or stone.

5. ***To find number of cubic yards of concrete,*** in any proportions, made from one barrel of cement, divide 27 by the number of barrels of cement per cubic yard, obtained as above.

Volume of Plastic Mortar made from Different Proportions of Cement and Sand.

Quantities of Materials per Cubic Yard. (See p. 227.)

Relative proportions by volume*		Volume of Compacted Plastic Mortar						Materials for 1 cu. yd. Compact Plastic Mortar Based on barrel of					
		from 1 cu. ft. Cement			from 1 bbl. Cement			3.5 cu. ft.		*3.8 cu. ft.*†		4 cu. ft.	
		Based on Portland Cement weighing			Based on barrel of								
Cement	Sand	108 lbs. per cu. ft.	*100 lbs. per cu. ft.*†	95 lbs. per cu. ft.	3.5 cu. ft.	*3.8 cu. ft.*†	4 cu. ft.	Packed Cement	Loose Sand	*Packed Cement*	*Loose Sand*	Packed Cement	Loose Sand
		cu. ft.	*cu. ft.*	cu. ft.	cu. ft.	*cu. ft.*	cu. ft.	bbl.	cu. ft.	*bbl.*	*cu. yd.*	bbl.	cu. yd.
1	0	0.93	0.86	0.80	3.2	3.2	3.2	8.31		8.31		8.31	
1	½	1.12	1.06	1.02	3.9	4.0	4.1	6.92	0.46	6.73	0.47	6.61	0.49
1	1	1.48	1.42	1.38	5.2	5.4	5.5	5.22	0.68	5.01	0.71	4.88	0.72
1	1½	1.84	1.78	1.74	6.4	6.7	7.0	4.20	0.81	4.00	0.84	3.87	0.86
1	2	2.20	2.14	2.11	7.7	8.1	8.4	3.51	0.91	3.32	0.93	3.21	0.95
1	2½	2.56	2.50	2.47	9.0	9.5	9.9	3.01	0.98	2.84	1.00	2.74	1.01
1	3	2.92	2.86	2.83	10.2	10.9	11.3	2.64	1.03	2.48	1.05	2.39	1.06
1	3½	3.28	3.23	3.19	11.5	12.2	12.8	2.35	1.06	2.20	1.08	2.12	1.10
1	4	3.64	3.59	3.55	12.8	13.6	14.2	2.12	1.10	1.98	1.11	1.90	1.13
1	4½	4.01	3.95	3.91	14.0	15.0	15.6	1.92	1.12	1.80	1.14	1.72	1.15
1	5	4.37	4.31	4.28	15.3	16.4	17.1	1.77	1.15	1.65	1.16	1.58	1.17
1	5½	4.73	4.67	4.64	16.6	17.7	18.5	1.63	1.16	1.52	1.18	1.46	1.19
1	6	5.09	5.03	5.00	17.8	19.1	20.0	1.52	1.18	1.41	1.19	1.35	1.20
1	6½	5.45	5.39	5.36	19.1	20.5	21.4	1.41	1.19	1.32	1.21	1.26	1.21
1	7	5.81	5.76	5.72	20.3	21.9	22.9	1.33	1.21	1.23	1.21	1.18	1.22
1	7½	6.18	6.12	6.08	21.6	23.2	24.3	1.25	1.21	1.16	1.22	1.11	1.23
1	8	6.54	6.48	6.44	22.9	24.6	25.8	1.18	1.22	1.10	1.24	1.05	1.24

NOTE. — Variations in the fineness of the sand and the cement, and in the consistency of the mortar may affect the values by 10% in either direction.

*Cement as packed by manufacturer, sand loose.

†***Use these columns ordinarily.***

Quantities of Materials for One Cubic Yard of Rammed Concrete.

Based on a Barrel of 3.5 Cubic Feet.

(See important foot-notes, also p. 225.)

Note. — Variations in the fineness of the sand and the compacting of the concrete may affect the quantities by 10% in either direction.

*Use 50% columns for broken stone screened to uniform size.

†Use 45% columns for average conditions and for broken stone with dust screened out.

‡Use 40% columns for gravel or mixed stone and gravel.

§Use these columns for scientifically graded mixtures.

Quantities of Materials for One Cubic Yard of Rammed Concrete. Based on a Barrel of 3.8 Cubic Feet.

(See important foot-notes, also p. 225.)

NOTE — Variations in the fineness of the sand and the compacting of the concrete may affect the quantities by 10% in either direction.

*Use 50% columns for broken stone screened to uniform size.

†***Use 45% columns for average conditions*** and for broken stone with dust screened out.

‡Use 40% columns for gravel or mixed stone and gravel.

§Use these columns for scientifically graded mixtures.

Quantities of Material for One Cubic Yard of Rammed Concrete.

Based on a Barrel of 4 Cubic Feet.

(See important foot-notes, also p. 225.)

PROPORTIONS BY PARTS			PROPORTIONS BY VOLUMES			Volume of mortar in terms of percentage of volume of stone	PERCENTAGES OF VOIDS IN BROKEN STONE OR GRAVEL														
							50%*			45%†			40%‡			30%§			20%§		
Cement	Sand	Stone	Packed Cement	Loose Sand	Loose Stone		Cement	Sand	Stone	Cement	Sand	Stone	Cement	Sand	Stone	Cement	Sand	Stone	Cement	Sand	Stone
			bbl	cu. ft.	cu. ft.	%	bbl	cu. yd.	cu. yd	bbls	cu. yd.	cu. yd.	bbl	cu. yd.	cu. yd.	bbl	cu yd.	cu. yd.	bbl	cu. yd.	cu. yd.
1		1	1		4	89	4.99		0.74	4.80		0.71	4.62		0.69	4.23		0.63	3.91		0.58
1		2	1		8	49	3.57		1.06	3.37		1.00	3.20		0.95	2.84		0.84	2.56		0.76
1		3	1		12	35				2.60		1.16	2.45		1.09	2.13		0.95	1.90		0.84
1		4	1		16	28										1.71		1.01	1.51		0.89
1		5	1		20	24										1.43		1.06	1.26		0.93
1		6	1		24	22										1.22		1.08	1.07		0.95
1		7	1		28	20													0.94		0.98
1		8	1		32	18													0.83		0.98
1		9	1		36	17													0.75		1.00
1		10	1		40	16													0.68		1.01
1		11	1		44	15													0.62		1.01
1		12	1		48	15													0.57		1.01
1	1	1½	1	4	6	96	3.08	0.46	0.68	2.97	0.44	0.66	2.87	0.42	0.64	2.69	0.40	0.60	2.53	0.38	0.56
1	1	2	1	4	8	73	2.74	0.41	0.81	2.63	0.39	0.78	2.52	0.37	0.75	2.33	0.34	0.69	2.17	0.32	0.64
1	1	2½	1	4	10	59	2.47	0.37	0.91	2.35	0.35	0.87	2.25	0.33	0.83	2.06	0.31	0.76	1.90	0.28	0.71
1	1	3	1	4	12	50	2.25	0.33	1.00	2.13	0.32	0.95	2.03	0.30	0.90	1.85	0.27	0.82	1.70	0.25	0.76
1	1½	2	1	6	8	92	2.39	0.53	0.71	2.30	0.51	0.68	2.22	0.49	0.66	2.07	0.46	0.61	1.94	0.43	0.58
1	1½	2½	1	6	10	74	2.18	0.48	0.81	2.09	0.46	0.77	2.01	0.45	0.74	1.86	0.41	0.69	1.73	0.38	0.64
1	1½	3	1	6	12	62	2.01	0.45	0.89	1.91	0.42	0.85	1.83	0.41	0.81	1.68	0.37	0.75	1.56	0.35	0.69
1	1½	3½	1	6	14	54	1.86	0.41	0.96	1.77	0.39	0.92	1.68	0.37	0.87	1.54	0.34	0.80	1.42	0.32	0.74
1	1½	4	1	6	16	48	1.73	0.38	1.03	1.64	0.36	0.97	1.56	0.35	0.92	1.42	0.32	0.84	1.30	0.29	0.77
1	1½	4½	1	6	18	43	1.62	0.36	1.08	1.53	0.34	1.02	1.45	0.32	0.97	1.31	0.29	0.87	1.20	0.27	0.80
1	1½	5	1	6	20	39	1.52	0.34	1.13	1.43	0.32	1.06	1.35	0.30	1.00	1.22	0.27	0.90	1.11	0.25	0.82
1	2	3	1	8	12	74	1.81	0.54	0.80	1.74	0.52	0.77	1.67	0.50	0.74	1.54	0.46	0.68	1.44	0.43	0.64
1	2	3½	1	8	14	64	1.69	0.50	0.88	1.61	0.48	0.83	1.54	0.46	0.80	1.42	0.42	0.74	1.31	0.39	0.68
1	2	4	1	8	16	56	1.58	0.47	0.94	1.51	0.45	0.89	1.44	0.43	0.85	1.32	0.39	0.78	1.21	0.36	0.72
1	2	4½	1	8	18	51	1.49	0.44	0.99	1.41	0.42	0.94	1.34	0.40	0.89	1.23	0.36	0.82	1.13	0.34	0.75
1	2	5	1	8	20	46	1.40	0.42	1.04	1.33	0.39	0.98	1.26	0.37	0.93	1.15	0.34	0.85	1.05	0.31	0.78
1	2	5½	1	8	22	42	1.33	0.39	1.08	1.26	0.37	1.03	1.19	0.35	0.97	1.08	0.32	0.88	0.98	0.29	0.80
1	2	6	1	8	24	39	1.26	0.37	1.12	1.19	0.35	1.06	1.13	0.34	1.00	1.02	0.30	0.91	0.93	0.28	0.83
1	2½	3	1	10	12	86	1.65	0.61	0.73	1.59	0.59	0.71	1.53	0.57	0.68	1.42	0.52	0.63	1.33	0.49	0.59
1	2½	3½	1	10	14	75	1.55	0.57	0.80	1.48	0.55	0.77	1.42	0.52	0.74	1.32	0.49	0.68	1.23	0.46	0.64
1	2½	4	1	10	16	66	1.46	0.54	0.87	1.39	0.51	0.82	1.33	0.49	0.79	1.23	0.46	0.73	1.14	0.42	0.68
1	2½	4½	1	10	18	59	1.38	0.51	0.92	1.31	0.48	0.87	1.25	0.46	0.83	1.15	0.43	0.77	1.06	0.39	0.71
1	2½	5	1	10	20	54	1.31	0.48	0.97	1.24	0.46	0.92	1.18	0.44	0.87	1.08	0.40	0.80	0.99	0.37	0.73
1	2½	5½	1	10	22	49	1.24	0.46	1.01	1.18	0.44	0.96	1.12	0.41	0.91	1.02	0.38	0.83	0.93	0.34	0.76
1	2½	6	1	10	24	45	1.18	0.44	1.05	1.12	0.41	1.00	1.06	0.39	0.94	0.96	0.36	0.85	0.88	0.33	0.78
1	2½	6½	1	10	26	42	1.13	0.42	1.09	1.07	0.40	1.03	1.01	0.37	0.97	0.02	0.34	0.89	0.84	0.31	0.81
1	2½	7	1	10	28	39	1.08	0.40	1.12	1.02	0.38	1.06	0.96	0.36	1.00	0.87	0.32	0.90	0.79	0.29	0.82
1	3	4	1	12	16	75	1.35	0.60	0.80	1.30	0.58	0.77	1.25	0.56	0.74	1.15	0.51	0.68	1.08	0.48	0.64
1	3	4½	1	12	18	67	1.28	0.57	0.85	1.23	0.55	0.82	1.18	0.52	0.79	1.08	0.48	0.72	1.01	0.45	0.67
1	3	5	1	12	20	60	1.22	0.54	0.90	1.16	0.52	0.86	1.11	0.49	0.82	1.02	0.45	0.76	0.94	0.42	0.70
1	3	5½	1	12	22	55	1.16	0.52	0.95	1.11	0.49	0.90	1.06	0.47	0.86	0.97	0.43	0.79	0.89	0.40	0.72
1	3	6	1	12	24	50	1.11	0.49	0.99	1.06	0.47	0.94	1.01	0.45	0.90	0.92	0.41	0.82	0.84	0.37	0.75
1	3	6½	1	12	26	48	1.06	0.47	1.02	1.01	0.45	0.97	0.96	0.43	0.92	0.87	0.39	0.84	0.80	0.36	0.77
1	3	7	1	12	28	44	1.02	0.45	1.06	0.97	0.43	1.01	0.92	0.41	0.95	0.83	0.37	0.86	0.76	0.34	0.79
1	3	7½	1	12	30	42	0.98	0.44	1.09	0.93	0.41	1.03	0.88	0.39	0.98	0.79	0.35	0.88	0.73	0.32	0.81
1	3	8	1	12	32	39	0.94	0.42	1.11	0.89	0.40	1.05	0.84	0.37	1.00	0.76	0.34	0.00	0.69	0.31	0.82
1	4	5	1	16	20	75	1.08	0.64	0.80	1.03	0.61	0.76	0.99	0.59	0.73	0.02	0.55	0.68	0.86	0.51	0.64
1	4	6	1	16	24	63	0.99	0.59	0.88	0.95	0.56	0.84	0.91	0.54	0.81	0.83	0.49	0.74	0.77	0.46	0.68
1	4	7	1	16	28	55	0.92	0.54	0.95	0.88	0.52	0.91	0.83	0.49	0.86	0.76	0.45	0.79	0.70	0.42	0.73
1	4	8	1	16	32	48	0.86	0.51	1.02	0.81	0.48	0.96	0.77	0.46	0.91	0.70	0.42	0.83	0.64	0.38	0.76
1	4	9	1	16	36	43	0.80	0.47	1.07	0.76	0.45	1.01	0.72	0.43	0.96	0.65	0.39	0.87	0.60	0.36	0.80
1	4	10	1	16	40	40	0.75	0.44	1.11	0.71	0.42	1.05	0.67	0.40	0.90	0.61	0.36	0.90	0.55	0.33	0.81
1	5	10	1	20	40	47	0.70	0.52	1.04	0.66	0.49	0.98	0.63	0.47	0.93	0.57	0.42	0.84	0.52	0.38	0.77
1	6	12	1	24	48	46	0.59	0.52	1.05	0.56	0.50	1.00	0.53	0.47	0.94	0.48	0.43	0.85	0.44	0.30	0.78

NOTE. — Variations in the fineness of the sand and the compacting of the concrete may affect the quantities by 10% in either direction.

*Use 50% columns for broken stone screened to uniform size.

†Use 45% columns for average conditions and for broken stone with dust screened out.

‡Use 40% columns for gravel or mixed stone and gravel.

§Use these columns for scientifically graded mixtures.

√

Volume of Concrete Based on a Barrel of 3.5 *Cubic Feet.*

(See important foot-notes, also p. 225.)

PROPORTIONS BY PARTS			PROPORTIONS BY VOLUME			Volume of mortar in terms of percentage of volume of stone	AVERAGE VOLUME OF RAMMED CONCRETE MADE FROM ONE BARREL CEMENT				
							Percentages of Voids in Broken Stone or Gravel				
Cement	Sand	Stone	Cement	Sand	Stone		50%*	45%†	40%‡	30%§	20%§
			bbl.	cu. ft.	cu. ft.	%	cu. ft.	cu. ft.	cu. ft.	cu. ft.	cu. ft.
1		1	1		3.5	101	5.1	5.3	5.5	6.0	6.4
1		2	1		7.0	54	7.0	7.4	7.8	8.7	9.6
1		3	1		10.5	39		9.5	10.0	11.5	12.8
1		4	1		14.0	31				14.2	16.0
1		5	1		17.5	27				17.0	19.2
1		6	1		21.0	24				19.7	22.4
1		7	1		24.5	21					25.6
1		8	1		28.0	20					28.8
1		9	1		31.5	18					32.0
1		10	1		35.0	17					35.2
1		11	1		38.5	16					38.4
1		12	1		42.0	16					41.6
1	1	1½	1	3.5	5.2	104	8.0	8.3	8.6	9.1	9.7
1	1	2	1	3.5	7.0	78	8.9	9.3	9.7	10.5	11.2
1	1	2½	1	3.5	8.7	64	9.9	10.4	10.8	11.8	12.7
1	1	3	1	3.5	10.5	54	10.8	11.4	12.0	13.1	14.2
1	1½	2	1	5.2	7.0	95	10.2	10.6	11.0	11.7	12.5
1	1½	2½	1	5.2	8.7	78	11.2	11.6	12.1	13.0	14.0
1	1½	3	1	5.2	10.5	65	12.1	12.7	13.2	14.4	15.5
1	1½	3½	1	5.2	12.2	56	13.0	13.7	14.4	15.7	17.0
1	1½	4	1	5.2	14.0	50	14.0	14.8	15.5	17.0	18.5
1	1½	4½	1	5.2	15.7	45	14.0	15.8	16.6	18.3	20.0
1	1½	5	1	5.2	17.5	41	15.9	16.8	17.8	20.0	21.6
1	2	3	1	7.0	10.5	77	13.4	13.9	14.5	15.6	16.8
1	2	3½	1	7.0	12.2	67	14.3	15.0	15.6	17.0	18.3
1	2	4	1	7.0	14.0	59	15.3	16.0	16.8	18.3	19.8
1	2	4½	1	7.0	15.7	53	16.2	17.0	17.9	19.6	21.3
1	2	5	1	7.0	17.5	48	17.1	18.1	19.0	20.9	22.8
1	2	5½	1	7.0	19.2	44	18.1	10.1	20.2	22.2	24.3
1	2	6	1	7.0	21.0	41	19.0	20.2	21.3	23.6	25.8
1	2½	3	1	8.7	10.5	90	14.6	15.2	15.8	16.9	18.0
1	2½	3½	1	8.7	12.2	78	15.6	16.2	16.9	18.2	19.6
1	2½	4	1	8.7	14.0	68	16.5	17.3	18.0	19.6	21.1
1	2½	4½	1	8.7	15.7	61	17.5	18.3	19.2	20.9	22.6
1	2½	5	1	8.7	17.5	55	18.4	10.4	20.3	22.2	24.1
1	2½	5½	1	8.7	19.2	51	19.4	20.4	21.4	23.5	25.6
1	2½	6	1	8.7	21.0	47	20.3	21.4	22.6	24.8	27.1
1	2½	6½	1	8.7	22.7	44	21.2	22.5	23.7	26.2	28.6
1	2½	7	1	8.7	24.5	41	22.2	23.5	24.8	27.5	30.1
1	3	4	1	10.5	14.0	77	17.8	18.5	19.3	20.8	22.3
1	3	4½	1	10.5	15.7	69	18.7	19.6	20.4	22.1	23.8
1	3	5	1	10.5	17.5	62	19.7	20.6	21.6	23.4	25.3
1	3	5½	1	10.5	19.2	57	20.6	21.7	22.7	24.8	26.8
1	3	6	1	10.5	21.0	53	21.6	22.7	23.8	26.1	28.4
1	3	6½	1	10.5	22.7	49	22.5	23.7	25.0	27.4	29.9
1	3	7	1	10.5	24.5	46	23.5	24.8	26.1	28.7	31.4
1	3	7½	1	10.5	26.2	43	24.4	25.8	27.2	30.1	32.9
1	3	8	1	10.5	28.0	40	25.3	26.9	28.4	31.4	34.4
1	4	5	1	14.0	17.5	77	22.2	23.2	24.1	26.0	27.9
1	4	6	1	14.0	21.0	65	24.1	25.2	26.4	28.6	30.9
1	4	7	1	14.0	24.5	56	26.0	27.3	28.6	31.3	33.9
1	4	8	1	14.0	28.0	50	27.9	39.4	30.9	33.9	36.9
1	4	9	1	14.0	31.5	45	29.8	31.5	33.2	36.6	40.0
1	4	10	1	14.0	35.0	41	31.7	33.6	35.4	39.2	43.0
1	5	10	1	17.5	35.0	48	34.2	36.1	38.0	41.8	45.5
1	6	12	1	21.0	42.0	46	40.5	42.8	45.0	49.6	54.1

NOTE. — Variations in the fineness of the sand and the compacting of the concrete may affect the volumes by 10% in either direction.

*Use 50% column for broken stone screened to uniform size.

†Use 45% column for average conditions and for broken stone with dust screened out.

‡Use 40% column for gravel or mixed stone and gravel.

§Use these columns for scientifically graded mixtures.

Volume of Concrete Based on a Barrel of 3.8 Cubic Feet.

(See important foot-notes, also p. 225.)

PROPORTIONS BY PARTS			PROPORTIONS BY VOLUME			Volume of mortar in terms of percentage of volume of stone.	AVERAGE VOLUME OF RAMMED CONCRETE MADE FROM ONE BARREL CEMENT				
							Percentages of Voids in Broken Stone or Gravel				
Cement	Sand	Stone	Cement	Sand	Stone		50%*	***45%†***	40%‡	30%§	20%§
			bbl.	cu. ft.	cu. ft.	%	cu. ft.	***cu. ft.***	cu. ft.	cu. ft.	cu. ft.
1		1	1		3.8	94	5.3	5.5	5.7	6.2	6.7
1		2	1		7.6	51	7.4	7.8	8.2	9.2	10.2
1		3	1		11.4	36		10.0	10.6	12.2	13.6
1		4	1		15.2	29				15.2	17.1
1		5	1		19.0	25				18.2	20.6
1		6	1		22.8	22				21.1	24.0
1		7	1		26.6	20					27.5
1		8	1		30.4	19					31.0
1		9	1		34.2	18					34.4
1		10	1		38.0	17					37.9
1		11	1		41.8	16					41.4
1		12	1		45.5	15					44.8
1	1	1½	1	3.8	5.7	99	8.5	8.8	9.1	9.7	10.3
1	1	2	1	3.8	7.6	75	9.5	9.9	10.3	11.1	11.9
1	1	2½	1	3.8	9.5	61	10.5	10.0	11.5	12.6	13.6
1	1	3	1	3.8	11.4	51	11.5	12.2	12.8	14.0	15.2
1	1½	2	1	5.7	7.6	93	10.8	11.3	11.7	12.5	13.3
1	1½	2½	1	5.7	9.5	76	11.9	12.4	12.9	13.9	15.0
1	1½	3	1	5.7	11.4	64	12.9	13.5	14.1	15.4	16.6
1	1½	3½	1	5.7	13.3	55	13.9	14.6	15.4	16.8	18.2
1	1½	4	1	5.7	15.2	49	15.0	15.8	16.6	18.2	19.9
1	1½	4½	1	5.7	17.1	44	16.0	16.9	17.8	19.7	21.5
1	1½	5	1	5.7	19.0	40	17.0	18.0	19.1	21.1	23.2
1	2	3	1	7.6	11 4	75	14.3	14.9	15.5	16.7	18.0
1	2	3½	1	7.6	13.3	65	15.3	16.0	16.8	18.2	19.6
1	2	4	1	7.6	15.2	57	16.3	17.2	18.0	19.6	21.3
1	2	4½	1	7.6	17.1	51	17.4	18.3	19.2	21.0	22.9
1	2	5	1	7.6	19.0	47	18.4	19.4	20.4	22.5	24.5
1	2	5½	1	7.6	20.9	43	19.4	20.5	21.7	23.9	26.2
1	2	6	1	7.6	22.8	40	20.4	21.7	22.9	25.4	27.8
1	2½	3	1	9.5	11.4	87	15.7	16.3	16.9	18.1	19.3
1	2½	3½	1	9.5	13.3	75	16.7	17.4	18.1	19.6	21.0
1	2½	4	1	9.5	15.2	66	17.7	18.5	19.3	21.0	22.6
1	2½	4½	1	9.5	17.1	60	18.7	19.6	20.6	22.4	24.3
1	2½	5	1	9.5	19.0	54	19.8	20.8	21.8	23.0	25.9
1	2½	5½	1	9.5	20.9	49	20.8	21.9	23.0	25.3	27.6
1	2½	6	1	9.5	22.8	46	21.8	23.0	24.3	26.7	29.2
1	2½	6½	1	9.5	24.7	42	22.8	24.2	25.5	28 2	30.8
1	2½	7	1	9.5	26.6	40	23.9	25.3	26.7	29.6	32.5
1	3	4	1	11.4	15.2	76	19.1	19.9	20.7	22.4	24.0
1	3	4½	1	11.4	17.1	68	20.1	21.0	21.9	23.8	25.6
1	3	5	1	11.4	19.0	61	21.1	22.1	23.2	25.2	27.2
1	3	5½	1	11.4	20.9	56	22.1	23.3	24.4	26.7	28.9
1	3	6	1	11.4	22.8	52	23.2	24.4	25.6	28.1	30.6
1	3	6½	1	11.4	24.7	48	24.2	25.5	26.9	29.5	32.2
1	3	7	1	11.4	26.6	45	25.2	26.7	28.1	31.0	33.8
1	3	7½	1	11.4	28.5	42	26.2	27.8	29.3	32.4	35.5
1	3	8	1	11.4	30.4	40	27.3	28.9	30.6	33.8	37.1
1	4	5	1	15.2	19.0	76	23.9	24.9	25.9	28.0	30.0
1	4	6	1	15.2	22.8	64	25.9	27.2	28.4	30.8	33.3
1	4	7	1	15.2	26.6	55	28.0	29.4	30.8	33.7	36.6
1	4	8	1	15.2	30.4	49	30.0	31.7	33.3	36.6	39.9
1	4	9	1	15.2	34.2	44	32.1	33.9	35.8	39.4	43.1
1	4	10	1	15.2	38.0	40	34.1	36.2	38.2	42.3	46.4
1	5	10	1	19.0	38.0	47	36.9	38.9	41.0	45.1	49.2
	6	12	1	22.8	45.5	46	43.7	46.2	48.6	53.6	58.5

NOTE. — Variations in the fineness of the sand and the compacting of the concrete may affect the volumes by 10% in either direction.

*Use 50% column for broken stone screened to uniform size.

†***Use 45% column for average conditions*** and for broken stone with dust screened out.

‡Use 40% column for gravel or mixed stone and gravel.

§Use these columns for scientifically graded mixtures.

Volume of Concrete Based on a Barrel of 4 Cubic Feet.

(See important foot-notes, also p. 225.)

PROPORTIONS BY PARTS			PROPORTIONS BY VOLUME			Volume of mortar in terms of percentage of volume of stone	AVERAGE VOLUME OF RAMMED CONCRETE MADE FROM ONE BARREL CEMENT				
							Percentages of Voids in Broken Stone or Gravel				
Cement	Sand	Stone	Cement	Sand	Stone		50%*	45%†	40%‡	30%§	20%§
			bbl.	cu. ft.	cu. ft.	%	cu. ft.	cu. ft.	cu. ft.	cu. ft.	cu. ft.
1		1	1		4	89	5.4	5.6	5.8	6.4	6.9
1		2	1		8	49	7.6	8.0	8.4	9.5	10.5
1		3	1		12	35		10.4	11.0	12.7	14.2
1		4	1		16	28				15.8	17.8
1		5	1		20	24				18.9	21.5
1		6	1		24	22				22.1	25.1
1		7	1		28	20					28.8
1		8	1		32	18					32.4
1		9	1		36	17					36.1
1		10	1		40	16					39.7
1		11	1		44	15					43.4
1		12	1		48	15					47.0
1	1	1½	1	4	6	96	8.8	9.1	9.4	10.0	10.7
1	1	2	1	4	8	73	9.8	10.3	10.7	11.6	12.4
1	1	2½	1	4	10	59	10.9	11.5	12.0	13.1	14.2
1	1	3	1	4	12	50	12.0	12.7	13.3	14.6	15.9
1	1½	2	1	6	8	92	11.3	11.7	12.2	13.0	13.9
1	1½	2½	1	6	10	74	12.4	12.9	13.5	14.5	15.6
1	1½	3	1	6	12	62	13.5	14.1	14.8	16.0	17.3
1	1½	3½	1	6	14	54	14.5	15.3	16.0	17.6	19.1
1	1½	4	1	6	16	48	15.6	16.5	17.3	19.1	20.8
1	1½	4½	1	6	18	43	16.7	17.7	18.6	20.6	22.5
1	1½	5	1	6	20	39	17.8	18.9	19.9	22.1	24.3
1	2	3	1	8	12	74	14.9	15.6	16.2	17.5	18.8
1	2	3½	1	8	14	64	16.0	16.7	17.5	19.0	20.5
1	2	4	1	8	16	56	17.1	17.9	18.8	20.5	22.3
1	2	4½	1	8	18	51	18.1	19.1	20.1	22.0	23.9
1	2	5	1	8	20	46	19.2	20.3	21.4	23.5	25.7
1	2	5½	1	8	22	42	20.3	21.5	22.7	25.1	27.4
1	2	6	1	8	24	39	21.4	22.7	24.0	26.6	29.2
1	2½	3	1	10	12	86	16.3	17.0	17.6	18.9	20.2
1	2½	3½	1	10	14	75	17.4	18.2	18.9	20.5	22.0
1	2½	4	1	10	16	66	18.5	19.4	20.2	21.9	23.7
1	2½	4½	1	10	18	50	19.6	20.6	21.5	23.5	25.4
1	2½	5	1	10	20	54	20.7	21.8	22.8	25.0	27.2
1	2½	5½	1	10	22	49	21.8	22.9	24.1	26.5	28.9
1	2½	6	1	10	24	45	22.8	24.1	25.4	28.0	30.6
1	2½	6½	1	10	26	42	23.9	25.3	26.7	29.5	32.3
1	2½	7	1	10	28	39	25.0	26.5	28.0	31.0	34.0
1	3	4	1	12	16	75	20.0	20.8	21.7	23.4	25.1
1	3	4½	1	12	18	67	21.0	22.0	23.0	24.9	26.8
1	3	5	1	12	20	60	22.1	23.2	24.3	26.4	28.6
1	3	5½	1	12	22	55	23.2	24.4	25.6	28.0	30.3
1	3	6	1	12	24	50	24.3	25.6	26.9	29.5	32.1
1	3	6½	1	12	26	48	25.4	26.8	28.2	31.0	33.8
1	3	7	1	12	28	44	26.4	27.9	29.4	32.5	35.5
1	3	7½	1	12	30	42	27.5	29.1	30.8	34.0	37.2
1	3	8	1	12	32	39	28.6	30.3	32.0	35.5	39.0
1	4	5	1	16	20	75	25.0	26.1	27.2	29.3	31.5
1	4	6	1	16	24	63	27.2	28.5	29.8	32.4	35.0
1	4	7	1	16	28	55	29.3	30.8	32.4	35.4	38.4
1	4	8	1	16	32	48	31.5	33.2	34.9	38.4	41.0
1	4	9	1	16	36	43	33.6	35.6	37.5	41.4	45.3
1	4	10	1	16	40	40	35.8	38.0	40.1	44.4	48.8
1	5	10	1	20	40	47	38.7	40.0	43.0	47.3	51.7
1	6	12	1	24	48	46	45.9	48.5	51.1	56.3	61.4

NOTE. — Variations in the fineness of the sand and the compacting of the concrete may affect the volumes by 10% in either direction.

*Use 50% column for broken stone screened to uniform size.

†Use 45% column for average conditions and for broken stone with dust screened out.

‡Use 40% column for gravel or mixed stone and gravel.

§Use these columns for scientifically graded mixtures.

CHAPTER XIII

STRENGTH OF PLAIN CONCRETE

The strength of plain concrete, that is, of concrete without steel reinforcement, is governed primarily by

(1) The quality of the cement.

(2) The texture of the aggregate.*

(3) The quantity of cement in a unit volume of concrete.

(4) The density† of the concrete.

The percentage of cement and the density of the concrete, which are of special importance to the user in determining the proportions of materials, may be expressed more explicitly as follows:

(1) With the same aggregate the strongest concrete is that containing the largest percentage of cement in a given volume of concrete, the strength varying nearly in proportion to this percentage.

(2) With the same percentage of cement but different arrangement of the aggregates, the strongest concrete is usually that in which the aggregate is proportioned so as to give a concrete of the greatest density, that is with the smallest percentage of voids. In many cases relative densities nearly correspond to relative weights.

Although these laws have been long recognized in a general way, having been partially proved by experiments of Mr. John Grant as early as 1871, but few attempts have been made to apply them practically in the comparison of strengths of different mixtures of concrete.

The authors have evolved a formula (see p. 238) from which, knowing the exact quantities of the raw materials entering into a concrete of a certain strength, it is possible to estimate the approximate strength of any other concrete mixed in different proportions of the same materials, under similar conditions of manufacture, storage, age, and methods of testing.

The compressive fiber strength of concrete, which is an essential factor in the design of reinforced concrete, is proportional to the strength of concrete in direct compression.

The table of tests of beams on page 258 covers so wide a range of proportions that it may be employed for comparing the transverse strength of different mixtures.

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*The word aggregate is defined on page 1.

†The meaning of density is illustrated on pages 172 and 173.

Further information relating to the strength of concrete made from different materials and under various conditions is presented under separate headings in this chapter. The methods of making concrete specimens for testing are outlined on page 278.

COMPRESSIVE STRENGTH OF CONCRETE

The actual strength of concrete in compression, because of the limited capacity of testing machines, can be determined only by experiments upon comparatively small specimens from 4 to 12 inches square. The results from tests of such specimens are probably slightly lower than the actual strength of concrete in practice, carefully mixed and laid, because of the difficulty in obtaining homogeneous specimens. Experiments by the authors show that the strength of the same mixture tends to increase with the size of the specimen even if the relative dimensions remain constant. Of course carelessness or inexperience will produce irregular work in either actual or experimental construction.

The experimental strength of concrete is not always a criterion for fixing the proportions of mixture, in fact most concrete must be made stronger than the theoretical loading would require. A lean concrete, for example, although it may gain sufficient strength before the load is applied, may not be sufficiently strong at a short period to permit the removal of the molds or the ordinary wear during building, or for many purposes the lean concrete may be too porous. Often a lean Portland cement concrete may thus present no special advantage over a richer natural cement concrete. (See Chapter IV.)

Comparative Strength of Concretes of Different Proportions. The formula for strength of mortar derived by Mr. R. Feret and presented on page 141, as Mr. Feret himself states,* is not applicable to concrete. Our formula for concrete mixtures is therefore presented as a practical working formula of sufficient accuracy to compare the compressive strength of mixtures of the same materials in different proportions. Starting with the principles laid down in the two fundamental laws stated at the commencement of the chapter, it is evolved by trial by the method given on page 239, to fit the average results of a large number of tests made in this country and Europe.

Let

P = unit compressive strength of concrete.

c = absolute volume† of cement in a unit volume of concrete.

*Chimie Appliquée, p. 522.

†Method of determining densities and absolute volumes are described on page 135.

s = absolute volume of sand in a unit volume of concrete.

g = absolute volume of stone in a unit volume of concrete.

M = a coefficient, constant for all proportions of the same material mixed and stored under similar conditions, but varying with the texture of the coarse aggregate and the age of the specimen.

Then

$$P = M\left(\frac{c}{1 + c - (s + g)} - 0.1\right) \qquad (1)$$

The absolute volumes, as indicated on page 138, are really ratios of the actual volume of the concrete, representing the actual mass or total volume of solid particles in a unit volume of concrete. Since ratios are independent of the unit selected, the absolute units are the same for any system of measurement, and by changing the value of M the formula is adapted to English or Metric System. For example, if P expressed in terms of kilos grams per square centimeter requires a value of $M = 880$, P in pounds per square inch will require a value of $M = 880 \times 14.2^* = 12\ 500$. It follows that knowing for a given age the value of M and the strength of a concrete composed of known percentages of materials, it is possible to estimate the compressive strength at the same age of any other concrete of exactly known composition made under like conditions from similar materials, but differently proportioned.

A very slight variation in the values of the terms will so largely influence the result that the formula is only useful, on the one hand, where the specific gravities of the materials and the weights entering into a unit volume of concrete are determined so accurately that the absolute volumes can be calculated, and, on the other hand, for comparison of the strength of different mixtures of concrete under assumed average conditions. For the latter purpose the specific gravity of cement may be taken at 3.1 and of sand at 2.65, the weight of a barrel of cement as 376 pounds, the weight of the dry sand contained in a cubic foot of moist sand as 89 pounds, and the percentage of voids in the stone as 46%. In computations, values of absolute volumes must be carried to three places of decimals.

Now let

P' = compressive strength in pounds per square inch.

c_b = barrels of cement contained in a cubic yard of the concrete.

s_c = cubic feet of sand contained in a cubic yard of concrete.

g_c = cubic feet of stone contained in a cubic yard of concrete.

M' = a coefficient adapted to pounds per square inch.

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*See page 93.

Then from formula (1):

$$P' = M'\left\{\frac{c_b \dfrac{376}{193}}{1 + \dfrac{376}{193}c_b - 27\left(\dfrac{89}{165}s_c + 0.54g_c\right)} - 0.1\right\}$$

$$P' = M'\left\{\frac{c_b}{0.513 + c_b - 7.48\,(s_c + g_c)} - 0.1\right\} \qquad (2)$$

This formula, as stated above, is only adapted for average comparative determinations, or where the conditions exactly correspond to those assumed. It may be adapted to other sand and stone by altering the coefficients of s_c and g_c. The table on page 242 is based upon these formulas (1) and (2).

Formula (1) on page 238 is based upon the actual strength of concrete, as determined by tests of Mr. E. Candlot in France and those of several other authorities at the Watertown Arsenal, U. S. A. To illustrate its

FIG. 78.—Comparison of Authors' Formula with Tests of E. Candlot. (*See p.* 240.)

agreement with actual experiments, tests of Mr. Candlot upon broken stone and gravel concrete 28 days old, quoted in full on page 249, are plotted on the diagram, Fig. 78, page 239, and Mr. George A. Kimball's tests made at the Watertown Arsenal on specimens 6 months old in Fig. 79.

The accuracy of the formula is shown by the nearness of the points on each diagram to straight lines starting from the origin. The abscissa of each point is determined by calculation of the term in brackets in formula (1), and the ordinate is the actual breaking strength of the specimen at the given period. The value of M in each case is the tangent of the straight line drawn through the points. If Mr. Candlot's tests are plotted on cross-section paper and smooth curves of growth in strength drawn through

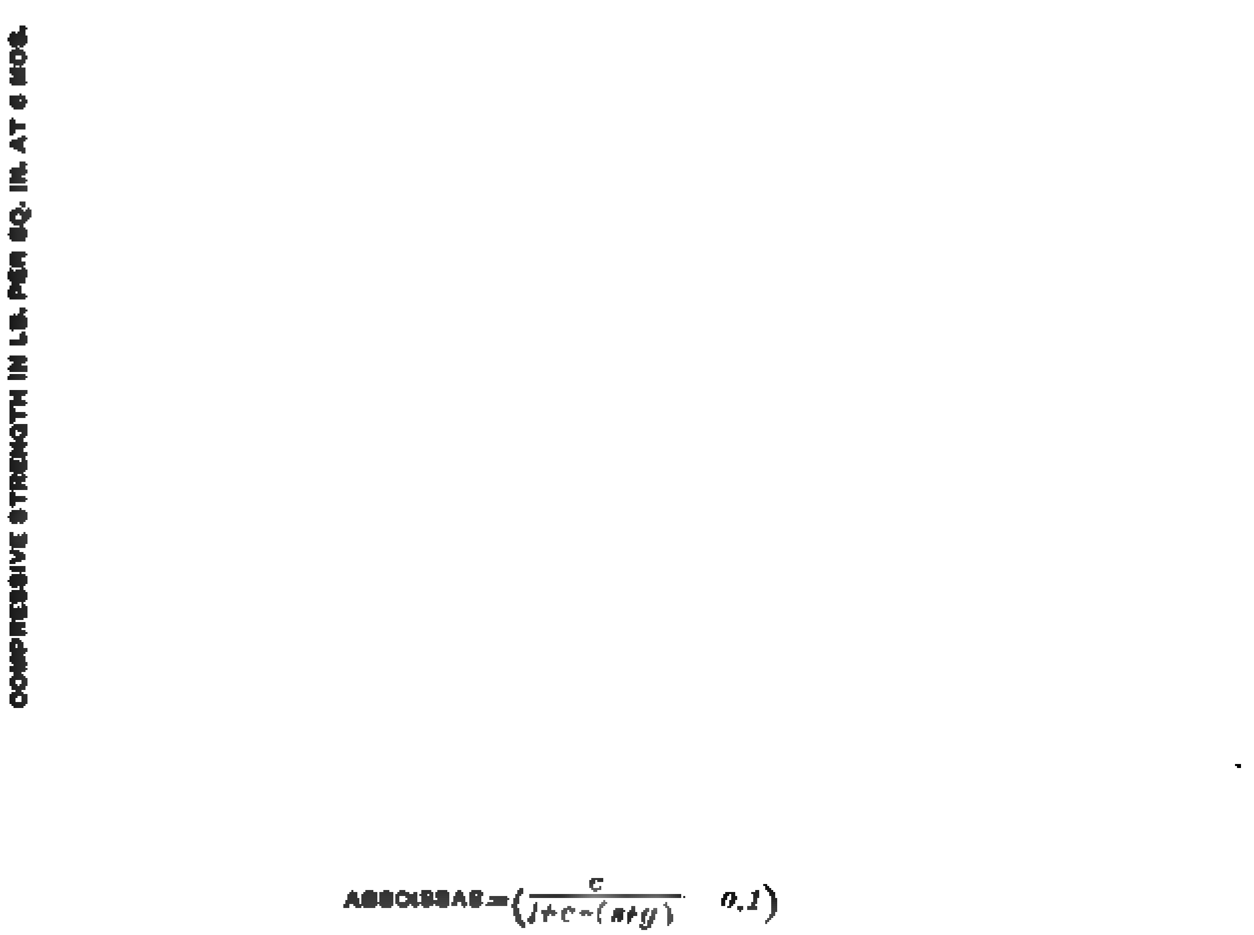

FIG. 79.—Comparison of Authors' Formula with Tests of George A. Kimball. (*See p.* 240.)

them, it will be found that the new values taken from such curves, which partially eliminate inequalities in the breaking, approach even more nearly to the straight lines.

After a study of the strength of concrete at different periods, the authors suggest the following values for M at different ages. The values for broken stone concrete are based upon stone ranging in size from 2 to 2½ inch down to ¼ to ½ inch. For broken stone of finer size the values will be slightly lower. The composition of the concrete does not affect the value of M, since the term of the formula in large brackets is itself dependent upon the proportions of the mixture and the density of the concrete. The values of M are directly proportional to relative strengths at different ages.

Value of Coefficient M for Compressive Strength in Pounds per Square Inch.

Age.	Coefficient M for broken stone concrete	Ratio of growth based on age at one month
7 days	9 500	0.76
1 month	12 500	1.00
3 months	15 600	1.25
6 months	16 900	1.35
1 year	18 000	1.44

The ratios, which are taken from the curve on page 257, are based on the assumption that growth in strength of concrete, mixed under similar conditions and of similar consistency, is the same for all proportions of like materials. This, as stated on page 256, is not strictly true, but is sufficiently accurate for practical purposes.

Table of Compressive Strength. The strength of concrete mixed in various proportions, given in the table on page 242, is based upon a strength with proportions 1: 3: 6, that is, one barrel cement to 11.4 cubic feet sand to 22.8 cubic feet stone, of 1950 lb. per square inch at the age of one month, this value being selected as the average of tests by different experimenters. It corresponds to a value of M of 12 500. Using 1950 lb. per square inch for 1: 3: 6 as the starting point, the strengths for other mixtures are calculated from formula (1) page 238, the absolute units for the different proportions being deduced from the average quantities of cement, sand, and stone, contained in a unit volume of concrete. The values employed are similar to those in the table on page 231, except that it was necessary to carry them to three places of decimals. The strength at the age of six months is based on the growth in strength given on the curve on page 257. The assumption, which corresponds to average conditions, is made that a cubic foot of moist bank sand contains 89 lb. of

dry grains having a specific gravity of 2.65, and that the specific gravity of the cement is 3.1. The cement is assumed to be first-class American Portland and the stone equal in quality to sound, hard limestone.

The values in the table may be readily transformed to safe working strength by dividing by the proper factor of safety. If concrete of special kinds of material mixed in certain proportions gives a higher or lower

Relative Compressive Strength of Portland Cement Concrete of Different Proportions.
(See important foot-notes, also p. 241.)

Proportions.			Age, one month.					Age, six months.				
			Voids in Broken Stone or Gravel.					Voids in Broken Stone or Gravel.				
Cement.	Sand.	Stone.	*50 % lb. per sq. in.	†45% lb. per sq. in.	‡40% lb. per sq. in.	§30% lb. per sq. in.	§20% lb. per sq. in.	*50% lb. per sq. in.	†45% lb. per sq. in.	‡40% lb. per sq. in.	§30% lb. per sq. in.	§20% lb. per sq. in.
1	1	2	2760	2740	2720	2680	2640	3720	3700	3670	3620	3570
1	1	3	2660	2630	2610	2550	2500	3590	3560	3520	3440	3380
1	1½	2	2880	2860	2840	2800	2760	3890	3870	3840	3780	3730
1	1½	3	2780	2750	2720	2670	2610	3750	3710	3680	3600	3530
1	1½	4	2680	2650	2610	2540	2460	3620	3570	3520	3430	3330
1	2	3	2560	2540	2510	2460	2410	3460	3420	3390	3320	3250
1	2	4	2480	2440	2410	2350	2290	3340	3300	3250	3170	3090
1	2	5	2400	2350	2310	2230	2170	3230	3180	3120	3010	2930
1	2	6	2320	2260	2230	2140	2060	3130	3060	3010	2890	2780
1	2½	3	2370	2340	2320	2270	2230	3200	3160	3130	3070	3020
1	2½	4	2290	2260	2230	2180	2110	3090	3050	3010	2940	2850
1	2½	5	2210	2180	2130	2070	2000	2980	2940	2880	2790	2700
1	2½	6	2140	2100	2060	1980	1910	2890	2830	2780	2670	2570
1	3	4	2120	2090	2060	2020	1970	2860	2830	2780	2720	2660
1	3	5	2060	2030	1990	1930	1870	2780	2740	2690	2610	2530
1	3	6	1990	1950	1910	1840	1770	2680	2630	2580	2480	2390
1	3	8	1860	1810	1770	1680	1600	2510	2440	2390	2280	2160
1	4	6	1710	1680	1650	1590	1530	2310	2270	2220	2140	2070
1	4	7	1660	1620	1590	1530	1460	2240	2190	2150	2060	1980
1	4	8	1610	1570	1530	1460	1400	2170	2120	2070	1970	1880
1	4	10	1510	1460	1420	1340	1260	2040	1980	1920	1810	1700
1	5	10	1310	1270	1230	1160	1090	1770	1720	1660	1570	1470
1	6	12	1060	1020	980	910	840	1430	1380	1320	1230	1140

NOTE.—Proportions are based on a barrel of 3.8 cu. ft. Values are for average ultimate strength, which must be divided by a factor of safety for working loads. Quality of materials and methods of mixing may affect the strength by 25% in either direction, while the relative values for different proportions are not materially changed.

*Use 50% columns for broken stone screened to uniform size.
†Use 45% columns for average conditions and for broken stone with dust screened out.
‡Use 40% columns for gravel or mixed stone and gravel.
§Use these columns for graded mixtures.

strength than that presented in the table, mixtures of these same special materials in other proportions may be assumed with approximate correctness to produce relatively higher or lower strengths than the tabular figures.

A point in the table which will appear inexplicable to users of concrete who have not carefully studied the true causes of strength in concrete is the fact that with the same proportions of mixture, the stronger concrete results with the stone having the larger percentage of voids. In explanation of this, it must be remembered that a material with a small percentage of voids contains in a unit volume, measured loose, a larger quantity of actual solids than a material with a larger percentage of voids. For example, stone with 40% voids has 60% of its bulk solid material, while one with 50% voids has 50% of its bulk solid material. Now, each particle of solid material occupies space in the volume of concrete, and a given volume of loose stone with 40% voids will therefore make more concrete if the voids are filled with mortar than the same loose volume of 50% stone mixed with the same volume of mortar. From table on page 234, we see that in the case of 1:3:6 concrete containing stone having 50% voids, one barrel of cement will make 23.2 cubic feet of concrete, while with the same proportions and stone having 40% voids, one barrel of cement will produce 25.6 cubic feet of concrete. Conversely there will be less cement in a unit volume of concrete with the stone having 40% voids. The density, on the other hand, will be but slightly increased, because, the same quantity of sand and cement being used, the particles of the stone containing the smaller percentage of voids are forced apart by the surplus mortar. The increase in density, in other words, is not sufficient to counterbalance the decrease in percentage of cement. If the proportions had been altered and the same percentage of cement, but less sand, used with the stone having 40% voids, the density of the concrete would have been greater than with the stone having 50% voids, and the per cent. of cement remaining the same, the concrete containing the stone with 40% voids would have been stronger than the other.

From this it must not be inferred that the aggregate with the largest percentage of voids is best to use. As indicated above, it requires more cement to a given volume of concrete, and the concrete is apt to be slightly less dense than with an aggregate having fewer voids, so that the latter is usually the more economical even although it is sometimes slightly inferior in strength. In the example in the preceding paragraph, with Portland cement at $2 per barrel, the concrete with stone having 50% voids would require 0.11 bbl. cement more per cubic yard than

the concrete with stone having 40% voids, and would therefore cost 22 cents higher per cubic yard.

Variation in Weight of Concrete of Different Proportions. The weights of specimens of similar concrete are of interest in comparing the relative strength of different mixtures or of different specimens of the same mixture. Of twelve pairs of duplicate cubes which the authors had tested in 1903 at the Watertown Arsenal and the Massachusetts Institute of Technology, the heavier specimen, except in one case, was found to be the stronger.

The following table of tests selected from tests of concrete and mortar cubes made by Mr. James E. Howard* at the Watertown Arsenal illustrates the comparative variation in weight and strength of concrete mixed in varying proportions:

Weights of Portland Cement Concrete of Different Proportions.

Age four months. Watertown Arsenal. (*See p.* 444.)

Item	Proportions by Volume			Weight per cu. ft.	Compressive strength per sq. in.
	Cement	Sand	Broken Stone†	lb	lb.
1	1	1	0	136.5	4370
2	1	2	0	134.2	2506
3	1	3	0	133.8	1812
4	1	4	0	120.9	830
5	1	5	0	119.3	532
6	1	6	0	116.9	169
7	1	7	0	111.5	118
8	1	2	4	150.7	2178
9	1	3	6	146.9	1815
10	1	4	8	143.2	1135
11	1	5	10	140.2	797
12	1	6	12	138.2	738
13	1	2	2	140.3	1768
14	1	2	3	145.2	1911
15	1	2	4	149.1	2147
16	1	2	5	150.9	2452
17	1	2	6	151.2	2124
18	1	2	7	146.4	1650
19	1	2	8	142.4	1295

Compressive Tests of Plain Concrete. The tests on pages 245, 249, and 248 (Fig. 81), are selected from among the best series of concrete experiments on record in America and Europe, so that the reader may form a general idea of the results obtained by expert experimenters. For practical comparisons of strength of different mixtures, reference should be made to the more complete table on page 242. The variation in strength of concretes mixed in the same proportions is due not only to the difference in the materials, but also to the different methods of making the tests, and to the fact that in many cases the unit of measurement

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*Tests of Metals, U. S. A., 1899, pp. 788–795.

†Items (8) to (12), 2½ inch screened broken trap, and items (13) to (19), 1½ inch screened broken trap.

Strength of Concrete in Compression from Various Authorities. Age, one month.

In pounds per square inch. (*See p. 244*)

[illegible]	Size of Specimen.	1:0	1:1	1:2	1:3	1:4	1:0.2	1:0.5	1:1:3	1:1½:4	1:2:2	1:2:3	1:2:4	1:2:5	1:2:7	1:2½:5	1:3:2	1:3:3	1:3:4	1:3:5	1:3:6	1:3:7½	1:4:3	1:4:5	1:4:8	1:5:10	1:6:13
Portland Cement Concrete.																											
¹Hawley & Krahl *a*	6"																				1705						
²Henby *b*	3" x 3½" x 11"												2800	2500							1900				1150		
³Humphrey *c*	6"																				1620						
⁴Luegers													2560								1990				1710	1350	1140
⁵Von Mauk	8"	4350	3560	3200	2560	1630					2060	1570					1570	1420	920				850				
⁶Dyckerhoff *d*	4"							1000				2800		2430						1660				1290			
⁷G. A. Kimball *e*	12"						3846						2300								2164						994
⁸Taylor & Thompson *f*	12"															3255											
⁹Woodb'y & Leight'n *g*	12"																			1623							
¹⁰Watertown Arsenal *h*	12"	5500	4400						4917											2042							
¹¹Webster	6"																				1763						
Natural Cement Concrete.																											
¹²Clarke & Son	12"									1440					746							991					
Cinder Concrete with Portland Cement.																											
¹³Watertown Arsenal	12"								1466			1098	904	769													
¹⁴Henby *i*	3½" x 3½" x 11"												1000	750							550	450					

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¹Engineering News, June [illegible]

1900, p. 145.
Vol XLVIII, p. 561.

⁹Tests at Watertown Arsenal 1903.

Sept., 1900, p. 145.

*a*Drift sand, blue limestone.
*b*¾" to 2" limestone.
*c*Gravel concrete.

*d*Stone 0.2" to 1.2" diameter.
*e*Average of 5 cements. Conglomerate stone.
*f*Newburyport sand ½" to 1" trap.

*g*Broken trap.
*h*1" trap.
*i*Cinders unscreened.

used in proportioning is indefinite, and, as discussed on page 218, similar nominal proportions may apply to quite different actual mixtures. Notwithstanding these opportunities for variation, however, it is noticeable that the results reached by different parties really show less percentage variation than is expected in the tensile tests of neat cements and sand mortars in different laboratories even with the same brand of cement.

Fig. 80. Twelve-inch Concrete Cube after Crushing in Emery Testing Machine at Watertown Arsenal. (*See p.* 247.)

In the table on page 245 of data from various authorities, only tests at the age of one month are recorded. Strength of the specimens at longer

and shorter periods may be estimated by referring to the curve in Fig. 84, page 257.

The appearance of a concrete cube after crushing, showing the manner in which the sides flake off, leaving a double pyramid, and the shearing of the particles of stone, is illustrated in Fig. 80. The specimen is one of a series tested for the authors at the Watertown Arsenal, U. S. A.

Kimball's Tests. A series of experiments upon 12-inch cubes made by Mr. George A. Kimball,* Chief Engineer of the Boston Elevated Railway Company, and tested at the Watertown Arsenal, although included in the above table, covers so wide a range in time and proportions that more complete values are worth quoting and are presented in the curves on page 248. Mr. Kimball also determined the elastic properties tabulated on page 266, and tested some of the specimens with a concentrated load, as referred to on page 250. He states that the stone used was conglomerate from Roxbury, Mass., containing 49.5 per cent. voids. Its analysis was as follows:

Passing	2½-inch ring	100.0%
"	2-inch "	95.2%
"	1-inch "	18.5%
"	½-inch "	0.5%

The sand and cement were made into a mortar of about the consistency of damp sand, and then spread upon the stone, which previously had been drenched with water. After ramming with iron rammers and tamping bars, the water barely flushed to the surface of the 1:0:2 and 1:2:4 mixture, while the surface of the 1:3:6 and the 1:6:12 mixtures appeared merely moist, so that the concrete was what ordinarily would be termed dry. The average quantity of water used with the different mixtures in addition to the water for wetting the stone is expressed in percentages of the weight of the cement and of the cement plus sand as follows:

Percentages of Water Employed in Kimball's Tests.

		In terms of weight of cement.	In terms of weight of cement plus sand.†
Mixture	1:0:2	20.9%	20.9%
"	1:2:4	30.3%	10.7%
"	1:3:6	39.3%	10.5%
"	1:6:12	71.1%	8.6%

These percentages *do not* include the water used in wetting the stone.

The specimens were made in cold weather, and therefore set slowly.

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*Tests of Metals, U. S. A., 1899, p. 717.

†Approximate.

They remained from two to seven days (most of them three to four days) in the molds, and were then placed, until tested, in wet ground. Mr. Kimball's remarks with reference to the leanest mixtures are of interest as illustrating the frequent necessity of using richer proportions than the actual loading requires.

> The 1:6:12 blocks were in poor condition. This was due to the difficulty of getting so lean a mixture well rammed into the corners of molds so small as 12-inch, and to the fact that the concrete had not attained sufficient strength, even though handled with care, to hold together well in the process of removal from the molds. The cubes of this mixture should have had a longer time to set before taking them out of the forms. In our foundation work we have used this mixture only as a filling with which to replace soft ground and on which to build the foundations proper.

The diagram in Fig. 81 shows Mr. Kimball's resultant curves* for the

FIG. 81.—Tests on Concrete Cubes by Geo. A. Kimball (Watertown Arsenal, 1889). (*See p.* 247.)

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*From data presented to the authors by Mr. Kimball.

different proportions based on an assumed weight of cement of 100 lb. per one cubic foot at the various ages. The results from individual brands of cements are shown by separate points.

Candlot's Tests. The table below, giving results of tests by Mr. E. Candlot,* of France, converted into English units, is of special value because of the accuracy in recording the data, the extreme variation in proportions and the number of periods at which specimens were crushed. The application of these tests to the authors' formula for strength is discussed on page 239.

Tests of Strength of Concrete made with Different Proportions.

By E. Candlot. (*See p.* 249.)

Proportions based on packed cement†	Volume of mortar in terms of percentage of volume of stone	Actual quantity of materials				Gravel concrete							Broken stone concrete						
		Cement	Sand	Stone	Water	Volume of Concrete	Cement in 1 cu. ft. of concrete	Weight per cu. ft. of concrete after setting	Ultimate Compressive strength in lb. per sq. in.				Volume of Concrete	Cement in 1 cu. ft. of concrete	Weight per cu ft of concrete after setting	Ultimate Compressive strength in lb. per sq. in.			
									7 Days	28 Days	6 Months	1 Year				7 Days	28 Days	6 Months	1 Year
	%	lb.	cuft	cu.ft.	cu.ft.	cu.ft.	lb.	lb.					cu.ft.	lb.	lb.				
1:6.4: 8.2	67	551	35.3	45.0	6.36	58.3	9.5	144.8	1031	1387	1280	1292	54.8	10.1	142.3	1316	1600	1636	1943
1:3.6: 4.7	67	992	35.3	46.6	7.42	61.1	16.2	147.3	1458	2454	2583	3225	56.9	17.4	147.9	2240	2845	3319	3508
1:2.5: 3.6	67	1433	35.3	50.9	8.55	65.0	22.0	150.4	2312	3094	3485	4385	61.1	23.4	149.8	2845	3485	4883	5026
1:1.6: 2.8	67	2205	35.3	62.0	10.77	78.0	28.0	149.8	2632	3414	3579	5500	72.0	30.6	151.6	3985	4303	4623	5974
1:6.4:10.9	50	551	35.3	60.1	6.36	67.8	8.1	142.3	747	924	1031	1707	63.6	8.7	142.3	1316	1387	1494	1683
1:3.6: 6.3	50	992	35.3	62.2	7.42	70.6	14.0	145.4	1743	1991	2536	2964	67.1	14.7	146.6	2098	2241	2845	3201
1:2.5: 4.7	50	1433	35.3	67.8	8.55	73.8	19.4	149.1	2169	3058	3532	4505	70.6	20.3	148.5	2276	3414	3627	5262
1:1.6: 3.7	50	2205	35.3	82.6	10.77	91.1	24.2	150.4	2952	3592	4054	5050	86.2	25.5	151.0	3556	3982	4338	5572
1:6.4:13.6	40	551	35.3	75.0	6.36	79.5	6.9	141.0	676	924	1078	1375	70.6	7.8	143.5	1280	1316	1138	1778
1:3.6: 7.8	40	992	35.3	77.7	7.42	84.8	11.7	142.3	1031	1494	1518	2608	78.8	12.6	142.3	1494	1778	2347	2822
1:2.5: 5.9	40	1433	35.3	84.8	8.55	90.4	15.9	145.4	1245	1992	2654	3247	85.5	16.7	146.0	2205	2525	2963	3201
1:1.6: 4.7	40	2205	35.3	103.3	10.77	106.7	20.7	149.2	2454	2560	3319	4503	102.4	21.5	146.6	2560	3200	3532	3936

Note. — The gravel weighed 96.8 lb. per cu. ft. and contained 40% voids. The broken stone weighed 85.5 lb. per cu. ft. and contained 47.4% voids. Both the gravel and broken stone had been passed through a screen having meshes of 1½" diameter. The sand weighed 81.2 lb. per cu. ft., thus containing 50.4% voids, and had been passed through a No. 12 sieve. The cubes were 10 centimeters (4 in.) on an edge.

The Effect of Concentrated Loading. In concrete foundations for piers and in concrete footings it is customary to load an area smaller than that of the surface of the concrete. The question at once arises whether the stress shall be based upon the load divided by the total area of the concrete footing or by the area of contact. Experiments made upon concrete and other materials show that neither of these methods is correct, but that an intermediate area should be selected for computation.

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*Candlot's Ciments et Chaux Hydrauliques, 1898, pp. 446, 447.

†Assuming 3.8 cu. ft. in 1 bbl of 376 lb.

In connection with the designing of concrete footings for the Boston Elevated Railway, 12-inch cubes were crushed by concentrating the load upon plates 10 by 10 inches and 8 by 8½ inches.*

In the diagram, Fig. 82,† is shown the relative strength of concrete under concentrated loads to that under distributed loading, and the curves are drawn, illustrating on the one hand the increased strength under concentrated loading if figured on the compressed area, and on the other hand the decreased strength if figured on the total area. These curves are similar in general direction, and also in the actual values of the ordinates, to curves drawn by Prof. J. B. Johnson‡ illustrating Bauschinger's tests upon other materials than concrete.

The method of using the curves shown in Fig. 82 is illustrated in the following examples: —

Example 1. — What dimensions of pedestal would be required to safely support a load of 20 tons concentrated upon a plate 10 inches square, assuming an allowable distributed stress upon the concrete of 350 lb. per square inch?

Solution.—Twenty tons or 40 000 lb. on 100 square inches represents 400 lb. per square inch,

Fig. 82.—Concentrated vs. Distributed Loading, by Geo. A. Kimball. (*See p. 250.*)

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*Tests of Metals, U. S. A., 1899, p. 740.

†From data presented to the authors by Mr. Kimball.

‡Johnson's Materials of Construction, 1903, p. 33.

a stress 14% greater than that allowed for distributed loading. Referring to the diagram, we find that for 1:2:4 concrete, 14% correspond to a ratio of areas of 0.68, hence the area of the concrete pedestal must be at least $\frac{100}{0.68} = 147$ square inches.

Example 2. — The breaking strength of a 12-inch cube of 1:2:4 concrete having chamfered edges so that the area of contact of the load is reduced to 9 by 9 inches, or 81 square inches, is 324 000 lb. What may be considered as the ultimate strength of the concrete when loaded over its full area?

Solution. — The strength per square inch of the cube, figured by its chamfered surface, is $\frac{324\,000}{81} = 4\,000$ lb. per square inch. From the upper curve in the diagram we find that where the ratio of the contact surface to the total area is $\frac{81}{144}$ or 0.56, the ratio of strength is 1.22. Dividing 4 000 lb., the unit strength on the concentrated surface, by this, gives 3 280 lb. per square inch as the ultimate strength of the concrete if it had been loaded over its full area.

The Strength of Short Prisms. The theoretical angle of rupture in crushing is about 60° with the horizontal, and as a matter of fact, cubes or prisms of concrete will leave, after crushing, pyramids whose surfaces are at an angle of about 60° with the base. To develop simply the normal compressive strength, the height of a specimen should be at least 1½ times, and preferably 5 times, its least lateral dimension.

The following formula evolved by Prof. Johnson* by plotting results of experiments by Prof. Bauschinger with sandstone prisms, and by Mr. Charles Bouton with cast iron prisms, may be used for comparing approximately the strength of prisms and cubes. Prof. Johnson states that the law holds between ratios of height to breadth of 0.4 to 5.0, the limits of the observations.

$$\frac{\text{strength of prism}}{\text{strength of cube}} = 0.778 + 0.222\frac{b}{h} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

where b = least lateral dimension of specimen,
and h = height of specimen.

Although we have not sufficient data to prove that this formula is exactly

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*Materials of Construction, 1903, p. 31.

applicable to concrete, a study by the authors of tests at the Watertown Arsenal* tends to show that, considering the variability of the material, it is probably sufficiently accurate for practicable use. In the Arsenal experiments square prisms were employed, varying in cross-section from 4 by 4 inches to 12 by 12 inches and ranging in height from 1 to 2 inches up to that of a cube. In every case the shorter prisms gave much higher strength than the cubes.

Example. — If the compressive strength per square inch of a 12-inch cube is 4 000 lb., what strength may be expected from a prism 12 inches square and 18 inches high?

Solution. — Substituting in formula (3), we have

$$\frac{x}{4000} = 0.778 + 0.222 \frac{12}{18}$$

$$x = 3704.$$

Theoretically, specimens of the same shape, as, for example, all sizes of cubes, should have the same strength per unit of area. In practice, large concrete cubes are apt to show higher unit strength than smaller ones; experiments by the authors, for example, giving in every case higher unit strength for 12-inch than for similar 8-inch cubes. However, the average unit weight of the 8-inch cubes was much lower than that of the 12-inch cubes made from the same batches of materials, indicating the difference in strength to be due to the fact that the materials can be more compactly placed in a large than in a small mold.

Plain Concrete Columns. There are few comparative records of the strength of concrete columns of different heights, but both theory and experiments tend to show that there is no appreciable difference in the compressive strength of columns of heights differing within ordinary limits, ranging, say, from a height of 3 to 14 times the least lateral dimension, provided the loading is exactly central. Prussian regulations† 1904 require that computation shall be made for flexure, if the height exceeds 18 times the least diameter. For reasons discussed in the preceding paragraphs the unit strength of cubes is greater than that of columns.

In the table which follows are given the results of tests of 12 by 12 inch columns of plain concrete, ranging from 2 to 14 feet in length, made by the Aberthaw Construction Company and crushed at the Watertown Arsenal.‡ Glancing at the columns headed "Percentage Variation from

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*Quoted and tabulated by Committee on Compressive Strength of Cements of the American Society of Civil Engineers in Transactions, Vol. XVIII, p. 264.

† See *Engineering Record*, July 2, 1904, p. 25.

‡Tests of Metals, U. S. A., 1897, p. 383.

the Mean," which we have added, it is noticeable that there is but very slight, if any, decrease in strength with the length of the specimen, and this difference may probably be due to the fact that the longer columns were tested at earlier periods. The ultimate strength per square inch of all of the tests is lower than the average records for similar proportions (see p. 242), because the specimens were made and kept at a temperature not far above freezing, but this does not affect the comparative values.

Compressive Strength of 12 by 12 inch Concrete Columns. (*See p.* 252.)

Made by ABERTHAW CONSTRUCTION COMPANY.

Nominal height ft.	Ratio of height to width.	Age tested days.	HAND-MIXED CONCRETE.		MACHINE-MIXED CONCRETE.	
			Strength in lb. per sq. in.	Percentage variation from the mean.	Strength in lb. per sq. in.	Percentage variation from the mean.
2	2	47	1072	+ 12.0	1185	+ 7.8
2	2	47	917	− 4.2	1183	+ 7.6
4	4	47	1067	+ 11.5	980	− 10.8
4	4	47	1132	+ 18.3	936	− 14.8
6	6	47	844	− 11.8	1131	+ 2.9
6	6	47	1048	+ 9.5	1200	+ 9.2
8	8	42	935	− 2.3	1108	+ 0.8
8	8	42	900	− 6.0	1086	− 1.2
10	10	41	909	− 5.0	1015	− 7.6
10	10	41	807	− 15.7	1000	− 9.0
12	12	39	947	− 1.0	1400	+ 27.4
12	12	39	980	+ 2.4	1500	+ 36.5
14	14	35	936	− 2.2	858	− 21.9
14	14	35	907	− 5.2	807	− 26.6
	Average		957	± 7.6	1099	± 13.2

Generally, the first sign of failure in the columns appeared in the form of longitudinal cracks, usually occurring from 0 to 2 feet distant from one end, although sometimes extending the entire length.

Eccentric Loading. The effect of eccentric loading, that is, of having the center of gravity of the load one side of the center of the column, is to lessen its compressive strength. A similar effect is produced by loading a column already bent, or by constructing it of unsymmetrical shape, as by bulging one side.

Most columns in actual structures are loaded more or less eccentrically, and this is especially the case with wall columns, which have all the floor loading upon one side. This must be allowed for in designing the columns.

The ordinary formula for the compressive fiber stress due to eccentric

loading upon solid rectangular columns, as illustrated in Fig. 83, is as follows:

Let

P = total load.
A = area of columns.
e = eccentricity.
b = breadth of column.
f = average unit pressure.
f' = total unit pressure on outer fiber nearest to line of vertical pressure.

Then

$$f' = \frac{P}{A}\left(1 + \frac{6e}{b}\right) \tag{4}$$

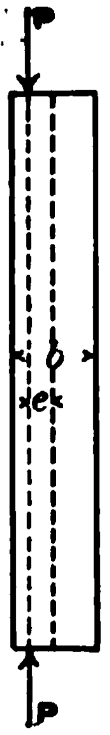

FIG. 83. Eccentric Column Loading. (*See p.* 254.)

The use of the formula is illustrated by the following example.

Example. — What will be the increase in pressure in a column 2 feet square due to placing the loading 6 inches off center?

Solution. — With central loading the pressure is, $f = \frac{P}{A}$

hence

$$f' = f\left(1 + \frac{6e}{b}\right)$$

Substituting the values $e = 0.5$ and $b = 2$

$$f' = 2\tfrac{1}{2}\, f$$

that is, the pressure on outer fibre is increased $2\frac{1}{2}$ times.

Concrete vs. Brick Columns. The compressive strength of brick piers is of interest to the concrete engineer for comparing brick and concrete columns. Tests made at the Watertown Arsenal and quoted by the Committee of the American Society of Civil Engineers on the Compressive Strength of Cement[*] give the ultimate strength of common brick piers about eighteen months old as ranging from 800 to 2 400 pounds per square inch, the results for brick laid with lime mortar averaging nearer the lower figure, and those for 1 : 2 Portland cement mortar nearer the higher figure.

Prof. William H. Burr,[†] after discussing the strength of brick piers under various conditions, states that

> The results of all the experimental investigations available in connection with brick masonry and experiences in the best class of engineering

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[*] Transactions American Society of Civil Engineers, Vol. XV, p. 717, and Vol. XVIII, p. 264.

[†] Burr's Materials of Engineering, 1903, p. 428.

> work indicate that masonry laid up of good hard-burnt common brick may safely carry a working load of 15 to 20 tons per square foot or 210 to 280 pounds per square inch. In the construction of this class of masonry where the duties are to be severe it is of the utmost importance that the best class of Portland cement mortar be employed, as the carrying capacity of brick masonry depends largely, if not chiefly, upon the character of the mortar.

These values are more than 20% lower than the requirements suggested on page 256 for columns of 1 : 2½ : 5 concrete, viz. 300 to 350 pounds per square inch, or 22 to 25 tons per square foot.

SAFE STRENGTH OF CONCRETE

Using experimental crushing tests as a basis, the safe working loads may be assumed to range from ⅓ to 1/10 of the breaking loads, depending upon the various conditions which are outlined below. Although these limits appear extreme, corresponding, for example, for 1 : 2½ : 5 concrete at the age of one month, to 730 to 220 pounds per square inch, different conditions will often warrant as great a variation in the selection of the unit pressure.

In many structures the actual strength of the concrete does not enter into the calculation. The dimensions of a concrete foundation, for example, are often determined by the area of the superimposed structure, or else, on the other hand, by the bearing power of the soil. In such cases it often would be theoretically possible to come nearer to the working strength of the concrete by using very lean proportions, were it not prohibited by the porosity of the mass or its low strength at short periods. However, by grading the materials so as to reduce the voids, a lean mixture is often economical.

The unit pressure to be selected depends not only upon the strength of the concrete as determined by its proportions, the character of the raw materials, and the methods of mixing, but also upon the character and importance of the structure, the nature of the pressure, — whether by direct compression or bending, whether from a live or dead load, or whether acting directly or through a cushion of inert material, — and the time of setting before placing the load.

The following values, while too arbitrary to satisfy all conditions, are given as fairly representing modern practice.

Safe Compressive Strength of Concrete.

CHARACTER OF PRESSURE	Safe strength at 1 month of 1:2½:5 mixture.†	
	lb. per sq. in.	tons per sq. ft.
Direct compression on mass concrete................	400	29
Compressive stress in reinforced beams*............	625	45
Columns over 2 square feet in sectional area	350	25
" under 2 " " " " "	300	22
Bearing of iron on concrete, such as bridge seats.....	400	29
Cinder concrete in direct compression	150	11

Piers or mass concrete subjected to pounding or vibrating load may require factors of safety nearly double the figures given and thus much lower working values.

GROWTH IN STRENGTH OF CONCRETE

Records from various tests made upon similar specimens of concrete at different periods are plotted in the diagram, Fig. 84. The curve illustrates the growth in strength which may be expected in ordinary average concrete made with first-class materials. The ordinates on the diagram represent ratios of the strength at various periods to the strength at the age of one month, in order that the curve may be of general application to various mixtures. If, for example, the strength of any concrete at one month is found to be 2 000 pounds per square inch, the strength of the same concrete at the age of six months may be assumed to be 2 000 multiplied by 1.35, the ordinate at six months, or 2 700 pounds per square inch.

The curve does not allow for the fact that the growth in strength varies to a certain extent with different materials, with different proportions, and with different percentages of water employed in mixing. As stated on page 272, with age, the strength of gravel concrete appears to gain on the strength of broken stone concrete. The growth, too, at periods beyond, say three months, is undoubtedly affected by the hardness or strength of the particles of the coarse aggregate, since a concrete of poor material will reach its ultimate strength earlier than one of good material. The tests of Mr. Kimball (see page 248) tend to show that the increase with age is greater with rich than with lean concrete, but on the other hand, tests of specimens made at the Watertown Arsenal indicate the reverse. The

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*When designed by the author's formulas, Chapter XIV.

†Proportions based on a barrel of 3.8 cubic feet, average strength of this mixture in simple compression being assumed as about 3000 lb. per square inch at the age of six months. (*See p.* 242.)

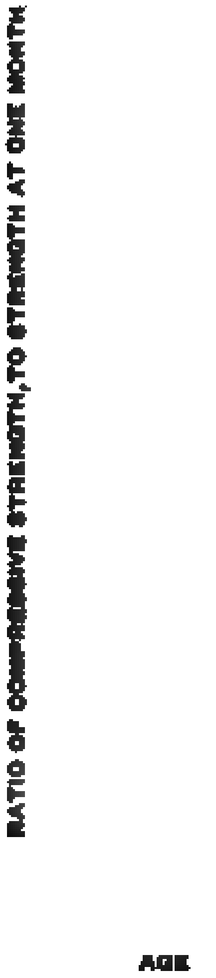

FIG. 84.—Growth in Compressive Strength of Portland Cement. (*See p.* 256.)

Data Concerning Composition and Transverse Strength of Concrete Beams Tested at Little Falls, N. J., by Wm. B. Fuller, C. E.

During the year 1901. Beams, 6 x 6 x 72 inches. Spans, 30 and 60 inches. Atlas Portland Cement, River Silica Sand. Crusher Run Trap Rock, ½ to 3 inches nominal diameter. (*See p.* 260.)

Item.	Proportions by weight.	Weight in Pounds of Material in one cu. ft. of Beam as Mixed.								Calculated Volume, in cu. ft. of Material in one cu. ft. of Beam as mixed.							Volume of Voids in one cu. ft.			Modulus of Rupture.					
							Totals.															Pounds per sq. in.			
	C.S.G.	Cement.	Sand.	Stone.	Total Dry Mix.	Water.	As Mixed.	Minimum.	Maximum	Cement.	Sand.	Stone.	Total Sand and Stone.	Total Dry.	Water, 62.4.	Total.	Cement.	Aggregate.	Total.	Age. Days.	Number of Breaks.	Maximum.	Minimum.	Average.	Per cent. probable error of average.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)	(24)	(25)	(26)
(1)	1:0:0	112.9			112.9	24.1	137.0	121.9	138.8	.585				5.85	.386	.971	.415	.000	.415	32	6	968	856	906	1.0
(2)	1:0:1	69.1		69.1	138.2	15.4	153.6	143.7	155.2	.358		.370	.370	.728	.247	.975	.254	.018	.272	32	6	872	668	772	2.8
(3)	1:0:2	49.9		99.8	149.7	12.9	162.6	153.7	162.6	.259		.534	.534	.793	.207	1.000	.183	.024	.207	33	6	802	668	731	2.4
(4)	1:0:3	38.0		113.8	151.8	11.2	163.0	154.8	163.9	.197		.609	.609	.806	.180	.986	.140	.054	.194	34	6	724	580	622	2.4
(5)	1:0:4	27.4		109.4	136.8	9.7	146.5	139.0	154.0	.142		.585	.585	.725	.155	.880	.101	.174	.275	34	3	251	236	241	2.3
(6)	1:1:0	64.9	64.9		129.8	15.3	145.1	135.0	146.7	.336	.393		.393	.729	.245	.974	.238	.033	.271	34	6	866	628	734	3.8
(7)	1:1:1	47.0	47.0	47.1	141.1	12.3	153.4	144.9	154.8	.244	.285	.252	.537	.781	.197	.978	.172	.047	.219	34	6	744	649	708	1.6
(8)	1:1:2	37.2	37.2	74.4	148.8	10.3	159.1	151.8	160.3	.193	.225	.398	.623	.816	.165	.981	.136	.048	.184	33	6	798	646	710	3.0
(9)	1:1:3	30.1	30.2	90.4	150.7	10.8	161.5	153.1	161.8	.156	.183	.483	.666	.822	.173	.995	.110	.068	.178	34	6	732	573	655	2.3
(10)	1:1:4	25.9	25.9	103.6	155.4	9.7	165.1	157.5	165.1	.134	.157	.554	.711	.845	.155	1.000	.095	.060	.155	33	6	512	446	486	1.9
(11)	1:1:5	22.6	22.6	113.0	158.2	7.8	166.0	160.0	167.1	.117	.137	.604	.741	.858	.125	.983	.083	.059	.142	34	6	542	481	504	1.6
(12)	1:2:0	43.5	86.9		130.4	12.9	143.3	133.9	145.9	.225	.527		.527	.752	.207	.959	.160	.088	.248	33	6	640	592	616	0.9
(13)	1:2:1	34.1	68.3	34.1	136.5	12.9	149.4	139.2	150.7	.177	.414	.182	.596	.773	.207	.980	.125	.102	.227	33	6	572	459	523	2.6
(14)	1:2:2	28.6	57.1	57.1	142.8	11.7	154.5	145.1	155.3	.148	.346	.305	.651	.799	.188	.987	.105	.096	.201	33	6	552	485	528	1.8
(15)	1:2:3	25.3	50.6	76.0	151.9	7.4	159.3	153.9	161.6	.131	.307	.406	.713	.844	.119	.963	.093	.063	.156	33	1			471	∞
(16)	1:2:4	22.3	44.7	89.4	156.4	7.4	163.8	158.2	164.8	.116	.271	.478	.749	.865	.119	.984	.082	.053	.135	33	6	480	399	439	3.0
(17)	1:2:5	19.8	39.5	98.5	157.8	7.4	165.2	159.4	166.0	.103	.239	.527	.766	.869	.119	.988	.073	.058	.131	33	6	413	349	380	2.2
(18)	1:2:6	17.5	35.0	105.1	157.6	8.2	165.8	159.0	166.0	.091	.212	.562	.774	.865	.131	.996	.064	.071	.135	35	5	410	234	319	9.5

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)	(24)	(25)	(26)
(19)	1:3:0	32.0	95.8		127.8	12.2	140.0	130.4	143.6	.166	.581		.581	.747	.196	.943	.117	.136	.253	35	6	432	302	418	1.2
(20)	1:3:1	27.4	82.3	27.4	137.1	10.9	148.0	130.3	150.4	.142	.490	.146	.645	.787	.175	.962	.101	.112	.213	35	5	392	274	360	3.3
(21)	1:3:3	21.0	63.2	63.1	147.3	8.6	155.9	140.0	158.0	.109	.383	.337	.720	.829	.138	.967	.077	.094	.171	33	6	369	338	355	1.2
(22)	1:3:5	16.6	40.9	83.2	149.7	8.0	157.7	151.0	160.1	.086	.302	.445	.747	.833	.128	.961	.061	.106	.167	33	3	308	262	285	3.3
(23)	1:3:6	15.3	45.9	91.9	153.1	8.5	161.6	154.3	162.6	.079	.278	.401	.769	.848	.136	.984	.056	.096	.152	33	5	246	213	220	2.1
(24)	1:3:7	14.3	42.8	100.0	157.1	8.2	165.3	158.2	165.3	.074	.259	.535	.794	.868	.132	1.020	.053	.079	.132	33	2	257	220	239	5.4
(25)	1:3:8	13.1	39.5	105.0	157.6	7.2	164.8	158.6	165.8	.068	.239	.561	.800	.868	.115	.983	.048	.084	.132	33	4	192	159	178	2.9
(26)	1:3:9	12.1	36.3	109.1	157.5	6.5	164.0	158.5	165.9	.063	.220	.583	.803	.866	.104	.970	.044	.090	.134	33	3	176	123	145	8.2
(27)	1:4:0	25.3	101.1		126.4	11.3	137.7	128.4	142.4	.131	.613		.613	.744	.181	.925	.093	.163	.256	33	6	204	262	270	1.8
(28)	1:4:2	18.9	75.7	37.9	132.5	12.8	145.3	134.0	147.5	.098	.459	.203	.662	760	.205	.965	.069	171	.240	34	4	235	198	210	3.1
(29)	1:4:4	15.8	63.0	63.1	141.9	10.7	152.6	143.2	154.3	.082	.382	.337	.719	.801	.171	.972	.058	.141	.199	34	3	219	202	209	1.8
(30)	1:4:6	13.6	54.2	81.4	149.2	10.0	159.2	150.3	159.6	.070	.328	.435	.763	.833	.160	.993	.050	.117	.167	34	2	184	114	149	16.7
(31)	1:4:7	12.7	50.6	88.6	151.9	9.5	161.4	152.9	161.4	.066	.307	.474	.781	.847	.153	1.000	.047	.106	.153	34	2	190	170	181	4.3
(32)	1:4:8	11.7	46.8	93.5	152.0	9.7	161.7	152.9	161.6	.061	.284	.500	.784	.845	.155	1.000	.043	.112	.155	34	2	158	156	157	0.4
(33)	1:4:9	11.1	44.3	99.8	155.2	8.7	163.9	156.1	163.9	.058	.268	.534	.802	.860	.140	1.000	.041	.099	.140	34	2	127	120	124	2.0
(34)	1:4:10	10.6	42.6	106.5	159.7	7.3	167.0	160.5	167.0	.055	.258	.570	.828	.883	.117	1.000	.039	.078	.117	34	2	133	130	132	0.8
(35)	1:5:0	20.9	104.7		125.6	13.0	138.6	127.3	141.6	108	.635		.635	.743	.208	.951	.077	.180	.257	33	4	180	170	173	1.0
(36)	1:5:3	15.5	77.5	46.5	139.5	10.8	150.3	140.7	152.0	.080	.470	.240	.719	.799	.173	.972	.057	.144	.201	34	2	153	140	151	0.9
(37)	1:5:5	13.4	67.1	67.1	147.6	10.2	157.8	148.7	157.9	.069	.407	.359	.766	.835	.163	.998	.040	.116	.165	34	2	163	159	161	0.9
(38)	1:5:7	11.8	58.8	82.3	152.9	8.9	161.8	153.8	161.8	.061	.356	.440	.796	.857	.143	1.000	.043	.100	.143	34	2	134	123	129	3.0
(39)	1:5:9	10.6	53.3	95.9	159.8	6.8	166.6	160.6	166.6	.055	.323	.513	.836	.891	.109	1.000	.039	.070	.109	33	2	113	105	109	2.6
(40)	1:5:11	9.3	46.7	102.6	158.6	7.5	166.1	159.3	166.1	.048	.283	.549	.832	.880	.120	1.000	.034	.086	.120	33	2	120	113	116	2.1
(41)	1:6:0	17.4	104.3		121.7	13.8	135.5	123.1	139.0	.090	.632		.632	.722	.221	.943	.064	.214	.278	33	2	94	92	93	0.8
(42)	1:6:2	14.5	[illegible]	29.1	130.7	12.4	143.1	131.9	145.7	.076	.528	.156	.684	.760	.199	.959	.053	.187	240	33	2	100	102	102	00
(43)	1:6:4	13.1		52.3	143.8	10.8	154.6	144.8	154.8	.068	.475	.280	.755	.823	.173	.996	.048	.129	.177	33	2	115	111	113	1.2
(44)	1:6:6	11.2		67.5	146.2	10.0	156.2	147.1	156.9	.058	.409	.361	.770	.828	.160	.988	.041	.131	172	33	1			78	00
(45)	1:6:8	10.2		82.0	153.7	8.5	162.2	154.5	162.2	.053	.373	.438	.811	.864	.136	1.000	.037	.099	.136	33	1			84	00
(46)	1:6:10	9.3		93.2	158.5	7.2	165.7	159.2	165.7	.048	.339	.498	.837	.885	.115	1.000	.034	.081	.115	33	2	91	87	89	1.6
(47)	1:7:0	15.6			124.8	12.8	137.6	126.0	140.8	.081	.662		.662	.743	.205	.948	.057	.200	.257	33	1			95	00
(48)	1:8:0	14.0			126.4	11.5	137.9	127.5	141.8	.073	.681		.681	.754	.184	.938	.051	.195	.246	33	1			41	00

Volumes, cu. ft. per 100 lb.
ft. as mixed,—cement,100;
of grains below diameter
c.08+Col. 6. Col. 10=Col.

cific gravity cement paste, 1.81; cemer
nt, lb. per sq. in., neat; 7 days, 834;
36; 50%, 0.028; 25%, 0.014; 0%,
m and maximum weights per cubic

2.99.
Analys
0%,

difference is slight in both cases, however, and it may be assumed for practical purposes that the rate of growth is approximately the same whatever the proportions. The consistency of the concrete, that is, the proportion of water used in mixing, affects the growth in strength to a certain degree, as described on page 270.

The curve does not apply to concretes of Natural cement mortar. 12-inch cubes of concrete in various proportions made from Akron Star cement tested at the Watertown Arsenal for William Wirt Clarke & Son* show an average ratio of increase in strength between one month and one year of 1.96. With this series of specimens the average strength at the age of one year was no greater than at seven months, but this is probably an exceptional case, since, for instance, tests by Capt. William M. Black on Natural cement concrete show a slower and continual growth, with an equally large ultimate strength.

TRANSVERSE STRENGTH OF CONCRETE

The strength of a beam of plain concrete is limited by the tensile strength of the concrete at the place of greatest strain, which, with vertical loading, is its lowest surface. The value of this transverse "fiber" strength or modulus of rupture is of less importance than the crushing strength, because, on account of the brittleness of concrete in tension, that is, its liability to crack from shrinkage or sudden loading, it is seldom safe, and usually is not economical, to construct beams or girders without metal reinforcement. Most formulas for reinforced design disregard the tensile strength of the concrete. In certain computations, however, the tensile strength must be considered. Since concrete beams can be broken with less powerful and less expensive apparatus than crushing specimens, this form of specimen is often convenient for comparing the relative strength of different mixtures or different materials, and while the ratios thus obtained will not exactly coincide with those for crushing strength, they will be sufficiently close for many purposes.

Fuller's Beam Tests. The table† on page 285 gives the results of a comprehensive series of tests of 6 by 6 by 72-inch beams made by Mr. William B. Fuller at Little Falls, N. J. Although different materials than those used by Mr. Fuller will of course show slightly different strength, the table is sufficiently representative of average conditions to permit its use for comparisons of different proportions, and, with a proper

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*Tests of Metals, U. S. A., 1901, p. 609.

†Especially prepared for this treatise by Mr. Fuller.

factor of safety, as a working guide to the safe transverse strength of concrete.

The proportions are given by weight but can be transformed to volume measure by referring to the footnote. The various columns present valuable data on weights and volumes and voids.

The curves in Fig. 85 are plotted from the results in the table, and illustrate also the proportions corresponding to maximum strength for a given per cent. of cement.

Tests by other authorities are mentioned under Strength of Beams in References, Chapter XXIX.

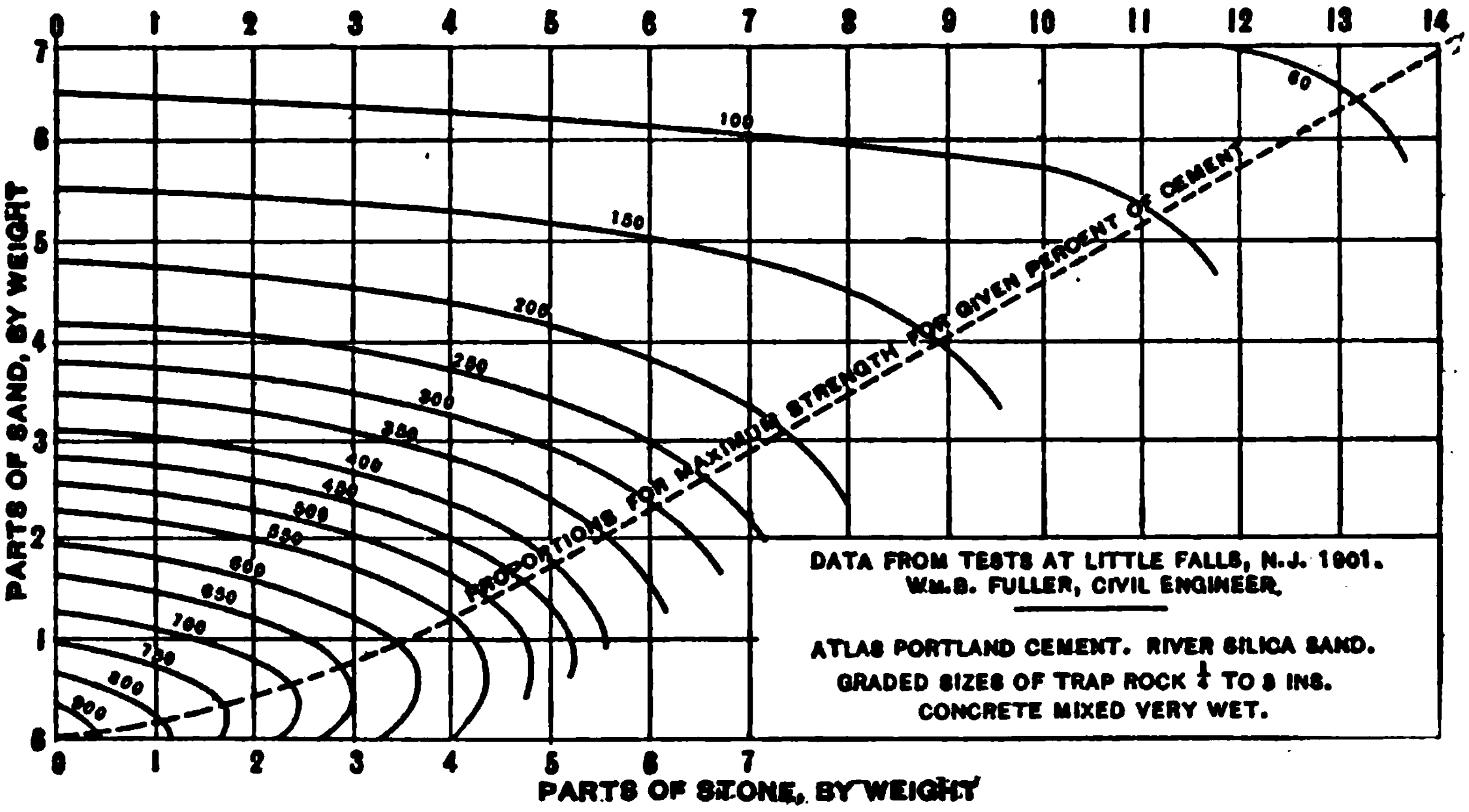

FIG. 85. Curves showing strength of beams in pounds per square inch for various proportions by weight of sand and stone to one part Portland cement.

Formula for Transverse or Bending Stress in Plain Concrete. The common formulas for representing the longitudinal forces of compression and tension upon a beam are usually expressed with the following notation:

Let

f = intensity of stress at any point in the beam.

M = bending moment.

I = moment of inertia about its neutral axis of section containing the point under consideration.

y = distance of the point from the neutral axis.

b = breadth of beam.

h = height of beam.

Then $f = \frac{My}{I}$ (5) also, $M = \frac{fI}{y}$ (6)

For rectangular sections, $I = \frac{bh^3}{12}$ and up to the elastic limit for beams of homogeneous material (but not for reinforced beams), $y = \frac{1}{2} h$.

Hence for rectangular beams of homogeneous material,

$$f = \frac{6M}{bh^2} \qquad (7) \qquad\qquad \text{also, } M = \frac{1}{6} f b h^2 \qquad (8)$$

In considering the strength of a beam, since the stress is greatest at one or the other of the surfaces, y is generally understood to represent the distance of the most strained fiber from the neutral axis, and f the intensity of stress upon this fiber.

The neutral axis — which is the line formed by the intersection of any cross section with the neutral plane, the plane upon which there is no longitudinal stress of either tension or compression — in a beam of homogeneous material passes through the center of gravity of the cross section. This is true for mortar and concrete which contain no reinforcement in the earlier stages of loading. Since, however, the neutral axis passes through the center of gravity of the beam only within the elastic limit,* the fiber stress, f, at the breaking point, as obtained by the common formula, does not represent the actual tensile stress upon the material. The comparative relations between different results, however, are unaffected by this limitation of the law, and the formula can therefore be used for comparing the strength of beams composed of similar material. For example, while the stresses at the instant of breaking, that is, the moduli of rupture, as figured by the formula, are not strictly correct either for 8 or 10 inch beams, they are nearly *proportional* to the actual stresses, so that the strength of plain concrete beams of different dimensions may be compared by means of the formula without appreciable error.

The following table for convenient reference gives values of the shearing force, S, and bending moment, M, for common cases, and the table on page 264 the moment of inertia,† I, for beams of a few sections which might be used in concrete construction. These tables are for the most part applicable to reinforced as well as to plain beams.

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*Although concrete and mortar have no true elastic limit the general principles apply to beams of these materials.

†Adopting notation in Lanza's Applied Mechanics.

Bending Moments and Shearing Forces.*

Description.	Loading.		Shearing Force. At distance x from origin.	Shearing Force. Greatest.	Bending Moment. At distance x from origin.	Bending Moment. Greatest.
Beam fixed at one end, free at the other.	At end		W	W	$W(l-x)$	Wl
	Uniform.		$\frac{W}{l}(l-x)$	W	$\frac{W}{2l}(l-x)^2$	$\frac{Wl}{2}$
Beam supported at both ends.	At middle.	Between origin and middle.	$\frac{W}{2}$	$\frac{W}{2}$	$\frac{W}{2}x$	$\frac{Wl}{4}$
		Beyond middle.	$-\frac{W}{2}$		$\frac{W}{2}(l-x)$	
	Uniform.		$\frac{W}{l}\left(\frac{l}{2}-x\right)$	$\frac{W}{2}$	$\frac{W}{2l}(lx-x^2)$	$\frac{Wl}{8}$
	At distance *a* from origin.	Between origin and load.	$\frac{W(l-a)}{l}$	$\frac{W(l-a)}{l}$	$\frac{W(l-a)}{l}x$	$\frac{Wa(l-a)}{l}$
		Beyond load.	$-\frac{Wa}{l}$	$\frac{Wa}{l}$	$\frac{Wa}{l}(l-x)$	

Relation of Transverse to Compressive Strength of Concrete. There is no fixed relation between the tensile fiber stress of concrete beams and the crushing strength of specimens made from the same material under identical conditions. The growth of strength is different in the two classes of tests, and although the general laws of increase in strength due to increasing the percentage of cement and the density appear to hold in both cases, the authors' formula given on page 238 for compressive strength is not applicable to transverse tests.

Experiments by the authors comparing 8-inch cubes and 8-inch beams of 1: 2½: 5 concrete give a ratio of crushing strength to modulus of rupture at one and two months of 6: 1.

Mr. A. Fairlie Bruce† states from his experiments on the strength of concrete bars and arch ribs that he found the ratio between the crushing strength of the arch and the modulus of rupture of the bars to be about

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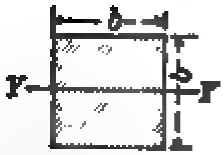
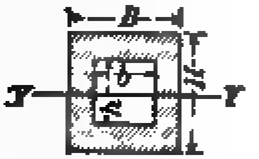
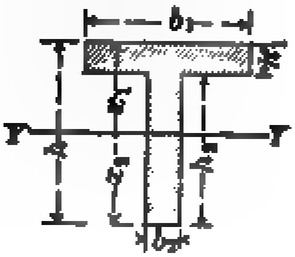
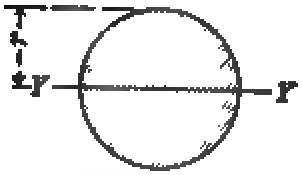
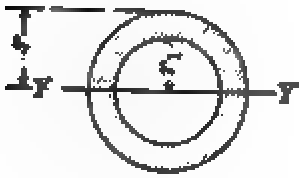
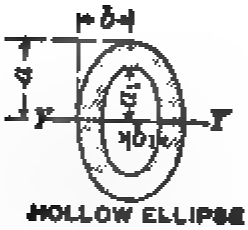
*W = total load; l = span; x = distance of load from origin.

†*Engineering Record*, Oct. 31, 1903, p. 533.

Moments of Inertia. (*See p. 262.*)

Figure.	Area A	Moment of Inertia. I	Distance of Neutral Axis from most Strained Fiber * Y
	bh	$\frac{bh^3}{12}$	$\frac{h}{2}$
 SQUARE	b^2	$\frac{b^4}{12}$	$\frac{b}{2}$
 SQUARE	b^2	$\frac{b^4}{12}$	$\frac{1}{2}b\sqrt{2}$
 RECTANGULAR CELL	$BH - bh$	$\frac{1}{12}(BH^3 - bh^3)$	$\frac{H}{2}$
 T SECTION	Area of flange + area of web $= A_1 + A_2$	$\frac{A_1h_1^2 + A_2h_2^2}{12}$ $+\frac{A_1A_2(h_1+h_2)^2}{4(A_1+A_2)}$	$x_1 = \frac{h}{2} - \frac{A_1h_2 - A_2h_1}{2(A_1 + A_2)}$ $x_2 = \frac{h}{2} + \frac{A_1h_2 - A_2h_1}{2(A_1+A_2)}$
 CIRCLE	πr^2	$\frac{\pi r^4}{4}$	r
 HOLLOW CIRCLE	$\pi(r^2 - r_1^2)$	$\frac{\pi(r^4 - r_1^4)}{4}$	r
 HOLLOW ELLIPSE	$\pi(ab - a_1b_1)$	$\frac{\pi a^3b}{4} - \frac{\pi a_1^3b_1}{4}$	a

*Applicable only to homogeneous (not to reinforced) beams.

6: 1 for concrete two to four weeks old, then increasing to about 10: 1 at the age of six months.

MODULUS OF ELASTICITY OF CONCRETE

The modulus of elasticity, that is, the stress or load at any point in the test divided by the total strain or deformation of the specimen at the same point (see page 267), is an important factor in the design of reinforced concrete. The value of the modulus increases with the age of the specimen and with the richness of the mixture. From experiments by Prof. C. Bach of Stuttgart, Germany, in 1895, summarized by Mr. David Molitor,* it would appear that the modulus of elasticity bears a definite relation, although not a fixed ratio, to the ultimate strength.

Different experimenters have reached exceedingly varied results in testing concrete for its modulus of elasticity. The differences, even in concrete composed of the same proportions of cement and aggregate, are often as great as from 1 500 000 to 5 000 000. The variation is due in part to the "personal equation" and the extreme delicacy required in measuring the deformation, and in part to differences in the quality of materials and in the methods of making and testing the specimens. Tests by Mr. Kimball,† tabulated on page 266 and described on page 247, present excellent records for 12-inch cubes, but as the gaged length for measuring the deformation of 12-inch cubes can be no more than 5 inches, and since, as indicated on page 267, the true measure of elasticity cannot be determined upon specimens of this shape, these results may not be accepted as conclusive unless confirmed by tests upon long prisms. Experiments by Prof. W. Kendrick Hatt‡ give values ranging from 3 500 000 to 4 000 000 for 1: 2: 4 mixture, and the results of Prof. W. H. Henby§ upon specimens 2¾ by 3½ inches by 11 inches long, give similarly high values. Recent tests upon long columns, the results of which have not yet been published, tend to confirm the lower values, such as were obtained by Mr. Kimball, for concrete of a character employed for reinforced construction.

Values for the modulus of elasticity of concrete of various proportions are suggested on page 285, where the subject is discussed from the standpoint of reinforced concrete design.

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*Journal Association of Engineering Societies, May, 1898, p. 348.

†Tests of Metals, U. S. A., 1899, p. 741.

‡Journal American Society for Testing Materials, 1902.

§Journal Association of Engineering Societies, Sept., 1900, p. 145.

Elastic Properties of Broken Stone Concrete 12-inch Cubes.

Portland cement,[*] bank sand and broken conglomerate stone.

By George A. Kimball at Watertown Arsenal. (*See p.* 265.)

COMPOSITION			Age	MODULUS OF ELASTICITY BETWEEN LOADS PER SQUARE INCH OF			Compressive strength per sq. in.
Cement	Sand	Broken Stone		100 and 600 lb.	100 and 1 000 lb.	1 000 and 2 000 lb.	lb.
1	2	4	7 days	2 593 000	2 054 000	1 351 000	1 730
1	2	4	1 mo.	2 662 000	2 445 000	1 462 000	2 567
1	2	4	3 mos.	3 671 000	3 170 000	2 158 000	2 975
1	2	4	6 mos.	3 646 000	3 567 000	2 582 000	3 989
1							
1	3	6	7 days	1 869 000	1 530 000		1 511
1	3	6	1 mo.	2 438 000	2 135 000	1 219 000	2 260
1	3	6	3 mos.	2 976 000	2 656 000	1 805 000	2 741
1	3	6	6 mos.	3 608 000	3 503 000	1 868 000	3 068
1							
1	6	12	1 mo.	1 376 000			1 146
1	6	12	3 mos.	1 642 000	1 364 000		1 359
1	6	12	6 mos.	1 820 000	1 522 000		1 592

Elastic properties of prisms of neat Portland cement and cement mortar, as made by Mr. Howard[†] at the Watertown Arsenal, are presented in the following table:

Elastic Properties of Cement and Mortar Prisms 6 by 6 by 18 inches.

Watertown Arsenal. (*See p.* 266.)

Brand of Cement	COMPOSITION		Age	MODULUS OF ELASTICITY BETWEEN LOADS PER SQUARE INCH OF			Permanent sets after loads per square inch of			Compressive strength per square inch.
	Cement	Sand	Days	100 and 600 lb.	100 and 1 000 lb.	1 000 and 2 000 lb.	600 Inch	1 000 Inch	2 000 Inch	lb.
Alpha	Neat	0	7	7 143 000	5 000 000	8 333 000	0.	0.	0.	4 783
			7	4 167 000	3 600 000	3 448 000	0.	0.	.0002	5 000
Alpha	1	1	15	3 125 000	2 812 000	2 326 000	-.0002	-.0002	.0007	3 846
			36	2 381 000	2 500 000	2 941 000	0.	.0002	.0012	4 763
			36	2 632 000	2 727 000	3 030 000	.0001	.0002	.0010	4 948
Alpha	1	2	15	1 724 000	1 475 000		.0005	.0023		1 376
			36	2 273 000	2 195 000	1 538 000	.0001	.0006	.0040	2 184
			38	2 778 000	2 812 000	2 325 000	0.	.0004	.0020	2 755

Gaged length, 10 inches.

[*] Atlas, Alpha, Germania, and Alsen.

[†] Tests of Metals, U. S. A., 1898, p. 666.

Prof. Gaetano Lanza suggests that for accurate determinations the length of a specimen should be five times its least lateral dimension, and that in no case should the specimen be shorter than one and a half times its least lateral dimension plus a length of 6 inches at each end to counteract the effect of the heads of the machine. For a specimen 8 inches square this would give a preferable length of 40 inches with a minimum of 24 inches.

Modulus in Tension. If the strength of concrete in tension is considered in the design of reinforced beams, the modulus of elasticity in tension is a factor of the formula.

Tests by Prof. Hatt* indicate that the modulus in tension is approximately equivalent to the compressive modulus. This is at variance with earlier tests, which usually showed a considerably lower modulus in tension, and further experiments are necessary in order to reach clear conclusions on this point.

Determination of Modulus of Elasticity. The *stress* upon any plane in a body which is acted upon by external forces is the force with which the particles upon one side of the plane act upon the particles upon the other side of the plane. For example, in a beam, a superimposed load, or the weight of the beam itself, exerts a force which is transmitted to the particles of the beam, and tends in the lower part of the beam to pull these particles apart, and in the upper part of the beam to press them together.

Let

E = modulus (or coefficient) of elasticity.
p = unit stress in pounds.
P = total load in pounds.
A = area of section of the body.
l = length of specimen in inches throughout which this elongation or shortening is uniform.
e = elongation or *shortening* in inches under a stress uniform throughout its length.

In a homogeneous body submitted to a uniformly distributed tension or pull over its entire cross-section, the intensity of stress or the unit stress on the cross-section is $p = \frac{P}{A}$. The *deformation* upon a body subjected to direct tension or compression, if its elongation or compression under the pull be uniform throughout its length, is this elongation or compression divided by the length of the specimen, or as a formula, Deformation $= \frac{e}{l}$.

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*Journal Association of Engineering Societies, June, 1904, p. 321.

The modulus of elasticity is the quotient of stress per unit of area divided by the deformation,

$$E = \frac{p}{\text{deformation}} \quad \text{hence} \quad E = \frac{Pl}{Ae} \tag{9}$$

The modulus of concrete may vary, becoming smaller near the breaking point, hence in stating a value for a modulus the loading to which it applies should also be given. In determining the value of e in compressive tests the permanent set must be deducted from the compression.

The following table* is selected at random from Watertown Arsenal tests of 12-inch cubes to illustrate the method of recording the loads and deformation when determining elastic properties:

Marks, *B*–9.
Composition: Atlas Portland cement, 1; sand, 3; broken stone, 6.
Age, 3 months.
Weight per cubic foot, 152.15 pounds.
Sectional area, 145.55 square inches.
Gaged length, 5 inches.

Applied Loads.		In gaged length.		Remarks.
Total.	Per square inch.	Compression.	Set.	
Pounds.	Pounds.	Inch.	Inch.	
14 555	100	0.	0.	Initial load.
29 110	200	.0001	0.	
43 665	300	.0004	0.	
58 220	400	.0006	.0001	
72 775	500	.0009	.0001	
87 330	600	.0011	.0002	E = 2 778 000 pounds per square inch
101 885	700	.0014	.0004	
116 440	800	.0017	.0005	
130 995	900	.0021	.0006	
145 550	1 000	.0025	.0007	E = 2 500 000 pounds per square inch
174 660	1 200	.0033	.0011	
203 770	1 400	.0044	.0016	
232 880	1 600	.0060	.0026	E = 1 471 000 pounds per square inch
261 990	1 800	.0080	.0036	
291 100	2 000	.0100	.0048	
320 210	2 200	.0122	.0064	
349 320	2 400	.0157	.0088	
370 000				First crack
378 430	2 600	.0196	.0116	
407 540	2 800	.0269	.0167	Ultimate strength

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*Tests of Metals, U. S. A., 1899, p. 754.

The modulus, $E = 2\ 778\ 000$, between loads of 100 and 600 lb. per square inch is calculated as follows:

$$\frac{P}{A} = 600 - 100 = 500$$

$$l = 5$$

$$e = 0.0011 - 0.0002 = 0.0009$$

$$E = \frac{Pl}{Ae} = 500 \frac{5}{0.0009} = 2\ 778\ 000$$

The modulus, $E = 2\ 500\ 000$ lb., is similarly determined between loads of 100 and 1 000 lb., and $E = 1\ 471\ 000$ between loads of 1 000 and 2 000 lb.

NUMBER OF REPETITIONS PRODUCING FAILURE

FIG. 86.—Fatigue of Neat Cement under Compression. (*See p.* 269.)

The modulus of elasticity may also be determined by drawing the stress-deformation curve as shown in Fig. 88, page 286, and measuring the tangent of its angle. This eliminates inaccuracies in measuring the deformations.

THE FATIGUE OF CEMENT

The action of cement under repeated stresses has been slightly investigated by Prof. J. L. Van Ornum* at Washington University. The experiments were made upon 2-inch neat Portland cement cubes four weeks old. The results of tests on 92 blocks are shown in the diagram in Fig. 86. Further investigation is required to determine the effect upon concrete of repeated applications of a load.

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*Transactions American Society of Civil Engineers, Vol. LI, p. 443.

STRENGTH OF CONCRETE IN SHEAR

Experiments on direct shear in concrete and mortar* are being conducted by Prof. Charles M. Spofford at the Massachusetts Institute of Technology. The results of the first series of tests indicate that the shearing strength of concrete and mortar is not far from one-half the strength in direct compression.

The specimens were 5 inches in diameter by 15½ inches long, and in testing were firmly held in cylindrical bearings 5½ inches apart, the load being applied from above through a half cylinder bearing, 5 7/16 inches in length. The stone in the concrete was Roxbury conglomerate mixed 3 parts 1-inch size, and one part ½-inch size. The proportions were based on a barrel of 3.5 cu. ft. After 24 hours in the molds, a part of the specimens were placed in water, a part in the air of the laboratory, and a part out of doors in moist sand. Apparently, the results were affected but little by these different treatments.

The table which follows gives the average results of the shearing tests, together with the crushing strength of 6-inch cubes of the same concrete for comparison of strength in shear and direct compression.

Shearing Strength of Concrete and Mortar.

By Charles M. Spofford. (*See p.* 270.)

		Shearing Strength.		Compressive Strength.	
Proportions.	Number of Specimens.	Age Days.	lb. per sq. in.	Age Days.	lb. per sq. in.
1:0	3	53	2753	40	4778
1:2	5	29	1331	33	4000
1:3	5	27½	839	38	1716
1:2:4	9	27	1082	33	2457
1:3:5	10	23½	560	24	1225
1:3:6	9	27½	612	27	1104

EFFECT OF THE CONSISTENCY UPON THE STRENGTH

The general result of experiments and practice tends to show that the strongest concrete can be secured with a mixture containing only sufficient water to produce a film of mortar upon the surface after very hard ramming in thin layers, but with a wetter "quaking" mixture the ultimate strength will be nearly as high as with the dry mixture, and because of the greater ease in laying and obtaining a homogeneous mass, it is generally to be preferred. An excess of water injures the cement by decomposing parts of it before it has had opportunity to set. The actual strength of concrete is often of less importance than other considerations. If, as in many classes of structures, there is an excess of strength, cheapness in placing, the ap-

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*Shearing tests of mortar, by Mr. Feret, are recorded on page 136.

pearance of the surface, or the proper imbedding of reinforcing metal, may be of primary importance. In such cases the quantity of water must be suited to the attendant conditions.

Tests by the authors indicate that (1) the consistency which will produce the densest concrete will result in the greatest ultimate strength, provided an excess of water is not employed; (2) dry mixtures attain higher strength at short periods, but mixtures of quaking consistency approach the dryer specimens after longer setting; (3) very wet mixtures, especially of lean proportions, may be chemically injured, but only to a slight extent, by the excess of water.

Effect of "Laitance." Whenever concrete is laid under water, the water is likely to be clouded by what appear to be particles of cement floating up from the mass which is being laid. This whitish substance is generally termed "laitance." A similar formation occurs on the surface of concrete laid with a large excess of water. In certain cases, we have found as much as ½ inch rising from a layer of 1: 2½: 5 concrete less than five inches thick.

Chemical and microscopical analyses, which Mr. Clifford Richardson has very kindly made for us, show that this laitance has nearly the same chemical composition,* except for a large loss on ignition, as normal Portland cements, but consists largely of amorphous material of an isotropic nature, — that is to say, it does not affect polarized light, and has almost no setting properties.

It is evident, therefore, that when concrete or mortar is laid under water, or with a large excess of water, a portion of the cement is rendered incapable of setting, and the strength of the mass is consequently reduced in proportion to this loss. The conclusion is naturally reached that for concrete laid under water, or in locations where a large excess of water is required in mixing, a higher percentage of cement than usual, about one-sixth more should be employed.

A lean mixture has been found to be more seriously injured by an excess of water than a rich one, probably because the water has a greater opportunity to penetrate the mass, and therefore to dissolve the cement.

GRAVEL VS. BROKEN STONE CONCRETE

Comparative tests of broken stone and gravel concretes, in the same proportions by volume, show almost invariably that concrete made from

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*See p. 393.

hard broken stone, such as trap, or hard limestone, gives higher compressive strength than concrete made from gravel. This appears to be the rule not only when the materials are mixed by measured volumes, regardless of the percentages of voids, but also when the broken stone and gravel are each screened to substantially the same sizes.

The relative values of gravel and broken stone concrete in the table which follows are based on the comprehensive series of comparative tests made by Mr. Candlot in France and tabulated on page 249.

Each ratio gives the extra strength of broken stone over gravel concrete of similar age. For example, if a concrete containing gravel having 40% voids tests 2 000 lb. per sq. inch at the age of six months, a concrete in similar proportions by volume containing broken stone with 47.4% voids should, according to Candlot's experiments, test 1.20 times greater or 2 400 lb. per sq. inch.

Comparative Strength of Broken Stone and Gravel Concrete.

From Candlot's Experiments

	Ratio of strength of broken stone concrete to gravel concrete.	
Age.	With equal voids	Broken stone 47.4% voids. Gravel, 40% voids.
7 days	1.30	1.33
1 month	1.26	1.19
6 "	1.18	1.20
1 year	1.12	1.09

The last column is averaged directly from Candlot's table, and may be taken as applicable to average conditions. It is noticeable that the gravel concrete approaches the broken stone concrete as its age increases. Since in many cases the ultimate strength of concrete is determined by the strength of its coarse aggregate, it follows that at, say, the age of a few months, a gravel concrete may reach or surpass the strength of a broken stone concrete having a coarse aggregate of soft stone of low strength.

Although the claim is frequently made that gravel concrete is stronger than broken stone concrete, the authors have failed to find substantial proof of this. On the other hand, various records, among them a number of tests at the Watertown Arsenal,* some of which are tabulated on page 274, tend to show the probable accuracy of Candlot's tests.

Another argument in favor of broken stone concrete lies in the fact that gravel is often covered with a film of dirt, difficult to remove, which lowers the strength. In experiments for the East Boston Tunnel† by Mr. Howard

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*Tests of Metals, U. S. A., 1898, pp. 649 to 654.

†Boston Transit Commission, 7th Annual Report, 1901, p. 39.

A. Carson, Chief Engineer, concrete beams made with washed gravel were about one-third stronger than beams made with gravel coated with a thin film of dirt.

Advocates of gravel concrete, among them Mr. R. Feret,* assert that as the rounded stones slip more readily into place, it is easier to make with them a compact mass. Loose rounded stones also contain a smaller percentage of voids than angular, but this is at least partly offset by the fact shown by the experiments of the authors tabulated on page 171, that broken stone compresses more on ramming.

Although the weight of evidence apparently favors broken stone concrete, it by no means follows that broken stone always should be used to the exclusion of gravel. In many instances, the ultimate strength of the concrete is of minor importance because the proportions of the concrete are determined by other considerations. Often, where strength is the criterion, but gravel is cheaper than broken stone, an additional percentage of cement may be economical. Moreover, the ultimate strength of gravel concrete is undoubtedly greater than that of concrete made with a poor quality of broken stone. With fixed proportions, as discussed on page 15, gravel is cheaper for the contractor than broken stone, because a given loose volume makes a larger quantity of concrete.

As indicated on page 243, in mixtures of like proportions by volume, the gravel concrete will have less cement in a cubic yard of concrete than a broken stone concrete unless the stone is well graded. Under ordinary conditions, to attain concretes of nearly equal strength, with gravel and with broken stone, the sand should be proportioned in each according to the volume and dimensions of the voids in the stone,† and the quantity of cement per unit volume of compacted concrete should be the same in each. The gravel concrete thus will be apt to be the denser, and this will tend to overcome the slight difference in strength due to the varying character of the surfaces of the particles of the gravel and broken stone.

Sometimes it is advantageous to mix a small percentage of gravel with broken stone.

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*Chimie Appliquée, p. 533.

†This can be better accomplished by trial mixtures, thoroughly compacted, of the dry aggregate or, still better, of small batches of concrete, than by water measurements of the voids. The proportions of the aggregates giving the smallest bulk of concrete to a given weight of the mixture of aggregates will be the best. Also, see Chapter XI on Proportioning.

EFFECT OF THE SIZE OF STONE OR GRAVEL UPON THE STRENGTH OF CONCRETE

The dimensions of the largest particles of stone and gravel which may be used in a concrete are often limited by practical considerations of mixing and placing. For ordinary work it is often specified that the stone shall pass through a 2-inch, or, more often, through a 2½-inch ring. For ordinary mass concrete of wet consistency the limit may be placed as high as 3 inches. In some cases, however, the stone must be small enough to pack readily around reinforcing metal, while in walls whose surface is to be picked or washed as described on page 381, a better appearance will result with stones under, say, one inch diameter, although the strength of concrete appears generally to increase with the size of the largest particles of stone in the mixture. This is illustrated with the gravel and the finer trap in the following table arranged from experiments by Mr. Howard* at the Watertown Arsenal upon 12-inch cubes of 1:1:3 concrete made with

Compressive Strength of Concrete made of Different Sized Stone, at different Ages.
Composition 1:1:3, Alpha cement.
Watertown Arsenal. (*See p.* 274.)

	FIRST GROUP			SECOND GROUP			THIRD GROUP			FOURTH GROUP		
KIND OF STONE	Age Days	Weight per cu. ft. lb.	Compressive strength per square inch lb.	Age Days	Weight per cu. ft. lb.	Compressive strength per square inch lb.	Age Days	Weight per cu. ft. lb.	Compressive strength per square inch lb.	Age Days	Weight per cu. ft. lb.	Compressive strength per square inch lb.
Trap ½"	7	145.56	1 391	19	149.00	2 220	32	146.44	2 800	76	153.34	5 021
Trap ¾"	8	147.01	1 900	20	150.12	2 769	32	148.27	3 200			
Trap 1 "	7	159.26	3 390	20	160.65	4 254	34	160.88	4 917	73	158.54	5 272
Trap 1½"	11	157.80	3 189	26	160.56	4 006	41 48	161.14 157.39	4 562 2 583			
Trap 2½"	7	158.37	2 400	22	159.27	4 143	32	161.44	4 140	65	161.76	4 523
Pebbles ⅜"	7	146.76	1 298	21	150.51	2 600	34	147.02	2 992	70	148.76	3 870
Pebbles 1½"	7	149.63	2 276	22	151.75	3 186	29	151.51	3 817	61	150.89	4 018
Pebbles 3 "	11	151.36	2 800	26	150.63	3 400	41 46	153.60 153.21	4 200 3 400			

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*Tests of Metals, U. S. A., 1898, p. 654.

uniform stone of different sizes. The weights of the specimens indicate that the increase of strength is due primarily to the density. The higher the limit of size the greater the variation in the sizes of material and therefore the greater the density of the mixture.

John Kyle* nearly doubled the strength of 1 : 2 : 6 concrete made with 1½-inch stone by substituting 4 parts of 3½-inch stone for a like portion of the 1½-inch.

EFFECT OF THE QUALITY OF THE STONE UPON THE STRENGTH OF CONCRETE

The ultimate strength of concrete is often limited by the texture or strength of the coarse aggregate. This is evidently the case with cinder concrete. Experiments by Mr. Geo. W. Raftert† gave the strength of concrete made with hard broken sandstone and various proportions of mortar from 1.5 to 2.4 times the strength of similar mixtures of broken shale and mortar, and this discovery led to the rejection of the latter as a material for concrete.

Tests of the authors upon 12-inch cubes broken at the Watertown Arsenal lead them to believe that at least in certain cases the ultimate strength of a concrete is actually fixed by the shearing strength of the particles of stone which make up the aggregate. Cubes in proportions 1 : 2½ : 4¾, — based on a cement barrel of 3.8 cubic feet, — attained an ultimate strength of 5000 to 5500 pounds per square inch. On account of differences in the methods of mixing and ramming, some of the specimens reached this limit at the age of two months while others did not attain it for six months; but it was noticeable that at whatever period the ultimate strength was reached the planes of fracture were smooth, breaking through each piece of stone, whereas before the ultimate strength was reached many of the stones pulled out from the concrete, leaving jagged instead of smooth surfaces on the pyramids remaining after the cubes were broken to destruction. The stone employed for these specimens was a hard, dense trap. If a weaker stone had been used, it is probable that the pieces would have sheared at a much earlier period and the ultimate strength would have been lower.

If concrete is mixed in such proportions or by such methods that the ultimate strength is reached before the stones shear, the strength of the particles of stone is a much smaller factor in the result.

Tests of crushing strength of building stone made by Mr. Richard L.

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*Proceedings Institution of Civil Engineers, Vol. LXXXVII, p. 88.

†Second Report on the Genesee River Storage Project, New York, 1894.

Humphrey* give the relative strength of specimens of several kinds of stone:

Crushing Tests of Cubes of Stone.

BY RICHARD L. HUMPHREY. (*See p.* 276.)

Location.	Kind of Stone.	Weight per cubic foot. lb.	Specific Gravity.	Absorption.	Average Crushing Strength.			
					2-inch cube.		8-inch cube.	
					Bed. lb. per sq. in.	Edge. lb. per sq. in.	Bed. lb. per sq. in.	Edge. lb. per sq. in.
Chester, Pa..........	Gneiss	165.71	2.69	0.385	6 097	5 446	9 505	6 426
Germantown, Pa.....	Gneiss	176.23	2.825	0.135	19 891	15 555	11 636	13 984
French Creek, Pa.....	Granite	190.46	3.085	0.155	19 997	14 348	17 274	7 910
Conshohocken, Pa....	Mica schist	177.76	2.91	0.155	20 038	15 680	10 417	7 532
Curwensville, Pa......	Sandstone	146.00	2.40	2.335	10 218	8 013	7 513	4 463
Lumberville, Pa......	Quartzite	158.19	2.63	0.998	no test	no test	14 841	8 637

The average of a large number of tests of 2-inch cubes, part on edge and part on bed, by Gen. Q. A. Gillmore, and quoted in Burr's "Materials of Engineering,"† shows average results for granite and sandstone almost identical with the average of Humphrey's tests on these materials, while the average strength of specimens of limestone and marble was about 13 000 lb. per square inch. Tests at the Watertown Arsenal‡ give the crushing strength of 4-inch cubes of sound trap rock as 33 300 lb. per square inch, and of seamy trap as 19 400 lb.

The table giving results of Mr. Humphrey's test is especially interesting as showing in a general way that the heaviest rock is apt to have the highest strength. Of the 8-inch cubes tested on their bed, so as partially to eliminate the effect of cleavage planes, the specimen of quartzite is the only one which does not follow this rule. In Gillmore's tests mentioned above, the variation in the same kind of stone from different localities is large, but in each kind the heavier rocks usually give the higher resistances. We may state, therefore, as a general rule in comparing rocks of the same kind, that those of the highest specific gravity are apt to be the strongest, and this rule may be extended in many cases to the comparison of different kinds of rock.

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*As tabulated by Edwin C. Eckel in *Engineering and Mining Journal*, June 20, 1903, p. 931.

†Edition of 1903, p. 433.

‡Tests of Metals, U. S. A., 1898, p. 577.

STRENGTH AND ELASTICITY OF CINDER CONCRETE

Tests at the Watertown Arsenal* on 12-inch cubes of cinder concrete mixed in different proportions gives results arranged in the following tables:

Compressive Strength of 12-inch cubes of Cinder Concrete.
Watertown Arsenal. (*See p.* 277.)

Cement.	Proportions Cement.	Sand.	Cinder.	Age, 1 month. Mean weight lb. per cu. ft.	Age, 1 month. Compressive strength lb. per sq. in.	Age, 3 months. Mean weight lb. per cu. ft.	Age, 3 months. Compressive strength lb. per sq. in.
German Portland..............	1	1	3	112.1	1 466	110.4	2 001
	1	2	3	115.2	1 098	112.8	1 634
	1	2	4	111.2	904	107.9	1 325
	1	2	5	108.8	769	105.3	1 084
	1	3	6	107.6	529	103.5	788
American Portland............	1	1	3	117.2	1 965	115.2	2 624
	1	2	5	111.3	818	110.0	1 412

Note: Each value for German cement is an average of three 12-inch cubes. Each value for American cement is an average of six 12-inch cubes made from two brands of first-class Portland cement. The exact age of the German cement specimens was 38 and 99 days, and of the American cement specimens 31 and 90 days.

Elastic Properties of Cinder Concrete, 12-inch cubes at three months.
Watertown Arsenal. (*See p.* 277.)

American Portland Cement.	Proportions.			Age when Tested.	Modulus of Elasticity between loads per sq. in.			Permanent sets after loads per sq. in. of			Compressive strength lb. per sq. in.
	Cement.	Sand.	Cinder.		100 and 600 lb.	100 and 1000 lb.	1000 and 2000 lb.	600 lb.	1000 lb.	2000 lb.	
	1	1	3	90	2 500 000	2 500 000	1 429 000	0.	.0001	.0006	2 780
A	1	2	5	90	1 087 000	957 000		.0008	.0028		1 402
	1	2	5	90	1 471 000	1 286 000		.0002	.0010		1 715
	1	1	3	90	4 167 000	3 214 000	1 190 000	0.	.0001	.0014	2 368
B	1	1	3	90	2 083 000	1 875 000	1 351 000	.0001	.0002	.0017	2 580
	1	2	5	90	1 190 000	849 000		.0009	.0066		1 200
	1	2	5	90	1 087 000	865 000		.0024	.0089		1 263

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*Tests of Metals, U. S. A., 1898, pp. 561 and 573.

MAKING CONCRETE SPECIMENS FOR TESTING

Complete and careful records must be made of the methods employed and the materials used in making concrete specimens for testing, in order to reach results of value for comparison with those of other experimenters. The lack of this care and accuracy has rendered the larger number of tests on concrete of only local significance.

The practical relation of the density of a concrete to its strength, as discussed in the preceding pages, indicates that it is not merely necessary to measure roughly the materials entering into the composition, but that the exact amount of solid matter, the coarseness of the particles, the character of the surfaces of the grains, the moisture in the materials, and the additional quantities of water used, must be very carefully recorded.

The blank form on page 281 is presented for recording data relating to the making of concrete specimens. Individual cases will require changes, but sufficient data should always be taken to ascertain the exact quantity of material entering into each specimen. In certain cases it is advisable for greater exactness to make separate batches for each specimen. The cost of making and testing concrete specimens is so great, that the additional time required for sufficient records to produce results of scientific value is insignificant.

The only observations required by the form which cannot be made with simple and inexpensive apparatus are the specific gravity of the cement and the mechanical analysis of the sand and stone. The specific gravity of Portland cement in most cases may be assumed as 3.1, and, in fact, the specific gravity of the sand may also be assumed without appreciable error as 2.65. If screens cannot be readily secured for mechanically analyzing the sand and stone, samples taken by methods of quartering described on page 281 may be sent to a laboratory for examination.

Concrete for experimental specimens should be mixed by experienced concrete laborers. There is a certain knack in properly turning the materials so as to mix them thoroughly which can be acquired only by practice, and the amount and manner of ramming or puddling is so important that specimens may be rendered worthless by improper manipulation.

The molds for specimens should be made from good quality lumber, preferably white pine, so that it will not twist or get out of shape, and the surface next to the concrete should be planed, and all joints made watertight. The mold should be wet or greased before placing the concrete. A mold for two cubes is shown in Fig. 87.

Dimensions of Specimens. Compression specimens are limited in size

by the capacity of the testing machine. The Emery Machine at the Watertown Arsenal, one of the largest in the world, has a capacity of 800 000 pounds, and the authors have had 12-inch concrete cubes tested there which reached this limit, so that 12 inches on a side may be fixed in general as the maximum size for specimens. For a lower limit it is doubtful if specimens less than 6 inches square can be made to give accurate results. A series of comparative tests by the authors upon 8-inch and 12-inch cubes gave much higher breaking strength per square inch for the larger size specimens. It was evident from the lower unit weight of the smaller specimens, that the difference was due, at least in part, to variation in homogeneity.

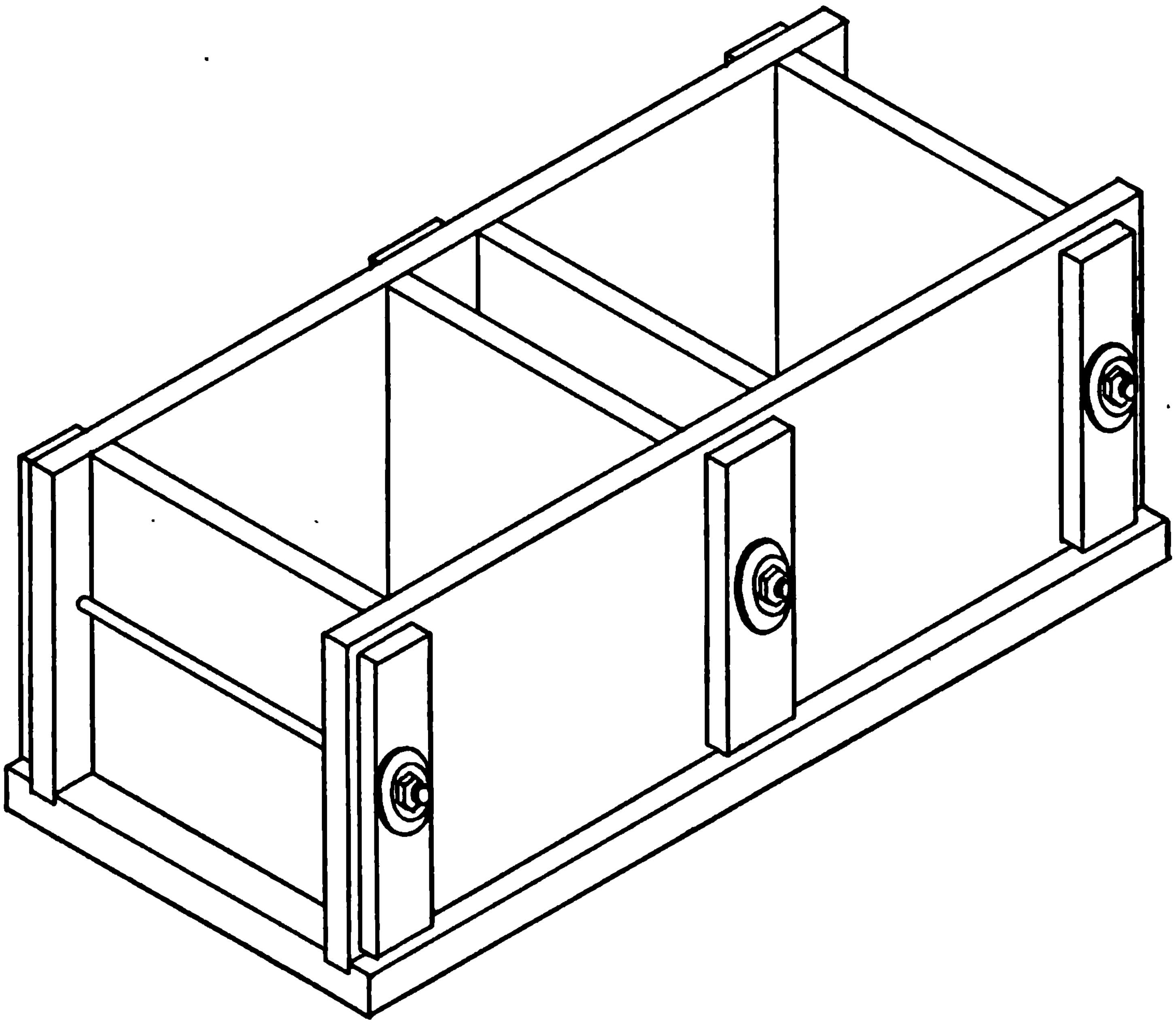

FIG. 87.—Mold for Concrete Cubes. (*See p.* 278.)

Cubes have been the common form of compression specimens and are suitable for comparative tests of ultimate breaking strength, but for studying the real value of concrete in compression, or for determination of elastic properties, long prisms, as described on page 267, are preferable.

For column tests, the length of a specimen should be at least five times the largest lateral dimension. Both theory and practice show that beyond this point there is but little variation in the strength per square inch, pro-

FORM FOR RECORDING DATA ON CONCRETE SPECIMENS

Place.. Experimenter.. Date.............................., 19...........

Cement, Brand...................... From...................... Specific gravity...................... Residue on No. 100 sieve..........%, on No. 200 sieve..........%.

Weight per cu. ft. (assumed)...................... ..lb. Remarks..

Sand,† Weight per cu. ft. (loose)...............lb. Moisture...............%. Specific gravity (dry)............... Mechanical analysis*: Passing sieves,

No. 10,...............%. No. 15,...............%. No. 20,...............%. No. 30,...............%. No. 40,...............%. No. 60,...............%.

No. 74,..........%. No. 100,..........%. No. 150,..........%. No. 200,..........%. Remarks..

Stone, No. 1.† Weight per cu. ft. (loose)..........lb. Specific gravity (dry).......... Voids (loose)..........%. Mechanical analysis: Passing sieves,

3",..........%. 2½",..........%. 1½",..........%. 1",..........%. 0.67",..........%. 0.45",..........%. 0.30",..........%. 0.20",...............%.

0.15",...............%. 0.10",...............%. Remarks..

Batch	Specimens	Proportions		% of water to weight of cement plus sand	Hour	Temperature of air	Cement	Sand†		Stone No. 1†		Water			Consistency	Volume				Weight							Disposition of specimens	REMARKS
																Green			Set	Green				Set				
		By weight	By volume				Weight	Weight	Volume	Weight	Volume	Temperature	Weight	Volume		Net	Surplus	Total	Net	Gross	Molds (wet)	Net	Net per cu. ft.	Date	Net	Net per cu. ft.		
		Parts	Parts	%			lb.	lb.	cu ft	lb.	cu.ft.		lb.	cu ft		cu ft	cu ft	cu ft	cu ft	lb.	lb.	lb.	lb.		lb.	lb.		

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*See page 187.

†Repeat for other numbers if sand or stone of more than two aggregates are used.

viding the loading is central. (See p. 253.) In tests for elasticity in compression, dimensions 8 by 8 by 24 inches are the smallest which the authors would suggest, and the results obtained from these before final acceptance should be verified by observation upon columns 12 inches square and several feet long.*

Beams for testing the transverse strength of concrete are usually made from 6 to 12 inches square. The smaller size is satisfactory provided the mixture is a fairly wet one so that the corners and surfaces of the molds can be filled. For specimens 6 inches square a convenient length is 6 feet, to be broken on a 60-inch span. The halves of the specimens may be afterwards broken to average with the full beam test or to compare the strength at different periods. Experiments prove that the ultimate fiber stress in the half beams will be practically, as well as theoretically, the same as that in the whole beams.

Specimens for crushing must be faced with some material which will transmit the strain to all points in the surfaces. At the Watertown Arsenal plaster of Paris or neat cement is employed. After spreading the surface with a coat of the plaster or cement, a block of polished steel is placed upon it, and it is allowed to set. Before crushing, the surface is tested with a straight-edge, and any irregularities are smoothed off with its sharp edge.

Method of Quartering. To obtain an average sample from a pile of sand, gravel, or stone, the method of quartering is useful. Shovelfuls of the material are taken from various parts of the pile, mixed together and spread in a circle. The circle is quartered, as one would quarter a pie, one of the quarters is shoveled away from the rest, thoroughly mixed, spread, and quartered as before. The operation is repeated until the quantity is reduced to that required for the sample.

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*See Discussion on Concrete by Sanford E. Thompson, International Engineering Congress, St. Louis, 1904.

CHAPTER XIV

REINFORCED CONCRETE

Reinforced concrete has been employed for only a few years, but the laws governing the combination of concrete and steel, although not absolutely fixed, are known with sufficient exactness to permit the design of nearly all classes of structures with the assurance — employing good materials and first-class superintendence — of permanent strength and durability.

The concrete itself plays a role as important as does the steel, and the variation in the strength and elasticity of this material under different conditions has sometimes been overlooked in the theoretical study of the combination of concrete and steel. This has led to apparent discrepancies in tests made in different localities, and has retarded the formulation of exact principles and laws of reinforced concrete.

To avoid confusion, only one of the numerous theories of distribution of internal stresses in a beam is presented in this chapter. For other theories and for the analysis of T-shaped beams and of beams having steel in the compression as well as the tension portion, the reader is referred to Appendix II.

In the following pages are discussed the principles governing the combination of steel and concrete (p. 283), the action of reinforced beams under load (p. 287), and the relative advantages of high and mild steel (p. 291). Working formulas for beam (p. 293) and for column design (p. 328) are presented, and tables are given for calculating the moment of resistance (pp. 302 and 309) and for determining the safe loading and reinforcement of beams and slabs (pp. 313, 317 and 318).

Vertical reinforcement of beams (p. 320), the required depth of concrete below the steel (p. 321), and the adhesion of concrete to the steel (p. 323) are discussed. Various experiments upon reinforced beams are referred to, and tests of Prof. Arthur N. Talbot are presented in detail (p. 326). Systems of reinforcement are briefly enumerated (p. 330).

Specifications for first-class or high carbon steel are given on page 38. The protection of metal from corrosion and fire is especially treated in Chapter XXI, and the design and building of reinforced structures in Chapters XXIII to XXVII. References to articles in engineering literature

treating of reinforced design and construction are tabulated in References, Chapter XXIX.

The term *stress*, which means the force, or load, (usually per unit of area) tending to stretch or shorten a body, is used in this chapter as little as possible, and the terms *pull* and *tension*, on the one hand, and *compression* or *pressure*, on the other, are substituted for it. The word *deformation*, that is, the shortening or stretching of the fibers of the concrete or steel, is employed in preference to strain.

GENERAL PRINCIPLES OF REINFORCED BEAMS

A concrete beam, when reinforced with iron or steel rods properly placed, develops a capacity for carrying loads several times greater than its carrying capacity when without reinforcement. It is evident that the location of the reinforcement in the beam must conform to the principles of mechanics so that the concrete shall be strengthened in its weakest part. Hence, since concrete is comparatively weak in its resistance to pull and to shear, the reinforcing metal should be placed where it will aid the concrete in carrying these stresses. In a beam or slab the metal should be as near to the surface on the tension side of the beam as is consistent with properly imbedding it and providing a sufficient thickness of concrete to protect it from rust and fire.

Since concrete is a brittle material and steel a comparatively ductile one, it might be expected that the stretching of the tension surface of a beam would result in the formation of unsightly and dangerous cracks on the under surface of the concrete, and that all the pull would be imposed upon the steel. Recent tests by Prof. Frederick E. Turneaure* indicate that cracks in the concrete are actually produced by the tension and that the tension load is thus transferred to the metal. However, while these cracks reduce the strength of the concrete, they are so minute, being at first invisible to the naked eye, and so distributed over the section, that the metal appears to be protected by the concrete at least up to the point of visible cracking, and probably beyond this point.

Not only is it essential to have the proper quantity of metal in the beam, but it must be correctly located. It is obvious that if the cross-section of the metal is too large as compared with the area of the concrete in compression, the beam, in case of failure, will give way by compression on the concrete, whereas, if the area of the metal is too small, weakness will show itself as soon as the metal has reached its yield point, which is usually not far from one-half the actual breaking strength of the steel. The area of

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*Proceedings American Society for Testing Materials, 1904.

the reinforcing metal in beams and slabs varies according to the conditions from about ¼% to 1½% of the area of the cross-section of the reinforced beam above the steel. For example, a beam 10 inches wide and 11 inches deep with steel one inch above bottom surface (100 square inches net area) requires, according to circumstances, from ¼ square inch to 1½ square inches section of steel. In any given design this area of reinforcement should be determined from the character of the member and the strength and elasticity of the concrete and the steel. More than 1% of steel is not usually economical unless the concrete is allowed to go beyond the high pressure of 750 pounds per square incn.

In designing a beam composed of concrete with steel imbedded in it, the bending moment produced by the superimposed load, — which is termed the live weight, — and the dead weight of the beam itself, must be no greater than the moment of resistance of the beam (*i.e.*, the moment of the internal resisting forces of the strength of the concrete and steel) divided by a proper factor of safety.

That which differentiates the study of a reinforced concrete beam from that of a beam composed of a single homogeneous material is the determination of the pull, which is borne by the steel alone, and of the compression, sustained entirely by the concrete. The problem is rendered the more complex because the strength and elasticity of concrete vary through a wide range according to the nature of its ingredients and their proportions. Current practice, however, which is borne out by recent experiments made at various American universities, indicates that beams may be designed on the assumption that the concrete in the upper part of the beam resists all the compression and the steel in the bottom of the beam takes all of the pull. The theories of the distribution of the stresses in reinforced concrete, which are based on the elasticity of the concrete and the steel, are sufficiently accurate for the practical purposes of design. Before giving formulas and tables to be used in the design of reinforced beams, the principles governing the assumption of the distribution of stresses in the properties of the materials will be considered.

A Plane Section Before and After Bending. Experiments by Prof. Talbot* at the University of Illinois indicate that the law holds for a reinforced concrete beam, as well as for beams of homogeneous material, that if a plane section is taken through a beam before loading, after loading, this section, even although inclined to its original position by the bending due to the load, remains a plane section. From this it follows, as in the common theory of beams, that the stretching or shortening per unit of

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*Proceedings American Society for Testing Materials, 1904.

length of any fiber which cuts the section considered, is proportional to the distance of this fiber from the neutral axis of the section.

Modulus of Elasticity. The modulus of elasticity of steel varies from 28 000 000 pounds per square inch to 31 000 000 pounds per square inch; 30 000 000 is customarily taken as an average value, and is the value which we have adopted.

The modulus of elasticity of concrete, a very important factor in reinforced concrete design, is considered in the preceding chapter, page 265. As there stated, it varies with the materials of which it is composed and with the proportions of these materials, also with the method of mixing and placing the concrete.

As tentative values for use in reinforced design, the authors suggest the following moduli for concrete mixed of the wet consistency usually employed in beams:

	Proportions.	Modulus of Elasticity lb. per sq. in.
Broken Stone or Gravel Concretes	1:1½:3	4 000 000
	1:2:4	3 000 000
	1:2½:5	2 500 000
	1:3:6	2 000 000
	1:4:8	1 500 000
Cinder Concrete................	1:2:5	850 000

It is probable that eventually these values will be found too low for dense, well-graded mixtures, which are gradually replacing those proportioned by rule of thumb methods. The authors have found a modulus of about 4 000 000 in 12-inch concrete cubes mixed 1: 2⅓: 4⅔, the crushing strength of which was about 5 000 pounds per square inch at the end of two months.

The higher the modulus of elasticity of the concrete, the lower should be the percentage of steel and the greater the depth of the beam for symmetrical design, that is, maintaining fixed relations of pull in steel to pressure in concrete.

From tests of Prof. W. Kendrick Hatt* the modulus of elasticity in tension appears to be of similar value to the compressive modulus. Earlier experimenters concluded that the modulus in tension is lower than in compression. A knowledge of the tensile modulus is, however, of less consequence than the other because the tensile resistance of concrete is not usually considered.

It is probable that there is an increase in the modulus of elasticity of concrete with age, but experiments by the author indicate that this is very slight.

*Journal Association Engineering Societies, June, 1904, p. 323.

Recent tests,* contrary to former ideas, indicate that under different loadings there may be but slight change in the modulus of elasticity of a given concrete until near to its crushing strength. This fact is of importance in fixing the distribution of stresses in the beam.

Relation of Stress-Deformation Curve to the Theory of Beams. A typical stress-deformation curve is shown in Fig. 88, which illustrates the compression of one of a series of 12-inch concrete cubes tested for the authors at the Watertown Arsenal.

The loads are represented as abscissas and the readings of the gage as ordinates. The upper line of circles represents the total or gross deformations in a gaged length of 5 inches and the lower line of circles the set when the load is removed. The stress-deformation curve, which follows the gross deformation curve as long as the set is zero, represents the gross deformation minus the set, that is, the elastic deformation. The tangent of the angle which the stress-deformation curve makes with the vertical line, multiplied by five (the gaged length in inches), gives the modulus of elasticity, which in the case considered is 3 840 000 pounds per square inch.

FIG. 88. — Elastic Deformation in Compression of 12-inch Cube of Plain Concrete. (*See p.* 286.)

The form of the stress-deformation curve of a compression specimen determines the distribution of the pressures in the portion of the beam which is under compression. If the stress-deformation curve is a straight line, the modulus of elasticity is a constant. It follows, from the assumption that a section plane before bending is plane after bending, that the

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*See Discussion on Concrete by Sanford E. Thompson, International Engineering Congress, St. Louis, 1904.

deformation, or stretch at any point in the compressive portion of the beam is proportional to the distance of this point from the neutral axis. If the modulus of elasticity is constant, the stress or pressure must be proportional to the deformation, and therefore the stress, or pressure, upon any fiber will be proportional to its distance from the neutral axis, and this stress, or pressure, may be represented by a straight line, as shown in Fig. 90, page 296. If the modulus of elasticity varies with the application of the load, the stress-deformation line becomes a curve.*

Elongation or Stretch in Concrete. According to tests of Prof. Turneaure, already mentioned, concrete under a pull, as in the lower portion of a beam, will usually stretch 0.0001 to 0.0002 of its length, that is, 0.01% to 0.02%, before showing minute cracks or "water-marks." Cracks become readily noticeable at a stretching varying, in different specimens, from 0.0003 to 0.0010 of their length. The concrete in a reinforced beam stretches similarly to the concrete in a plain beam except that in the latter the beam breaks when the limit of stretch is reached, while if reinforced, the pull is borne partly by the steel and partly by the concrete, and they both stretch together up to the point that cracks so minute at first as to be almost invisible occur in the concrete.

The action of the reinforced concrete is shown in the deflection curve in Fig. 89. The inclination of this curve changes at about the same load that is required to break a similar beam of plain concrete.

The diagram shows a typical result of Prof. Talbot's tests of the deformation of the concrete and the deformation of the steel, the deflection of the beam, and the various measured positions of the neutral axis during flexure. Among other conclusions, Prof. Talbot draws the following:

1. The composite structure acts as a true combination of steel and concrete in flexure during the first or preliminary stage, and this stage lasts until the steel is stressed to, say, 3 000 pounds per square inch, and the lower surface of the concrete is elongated about $\frac{1}{10\,000}$ of its length.
2. During the second or readjustment stage there is a marked change in distribution of stresses, the neutral axis rises, the concrete loses part of its tensional value, and tensile stresses formerly taken by the concrete are transferred to the steel. During this stage minute cracks probably exist, quite well distributed, and not easily detected.
3. In the third or straight-line stage the neutral axis remains nearly stationary in position and the concrete gradually loses more of its tensional value. Visible cracks appear and gradually grow larger, though no change in the character of the load-deformation diagram results. It would

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*A comprehensive analytical discussion of the effect of a varying modulus of elasticity upon the pressures in a beam under different loading is presented by Prof. Talbot in Journal Western Society of Engineers, August, 1904.

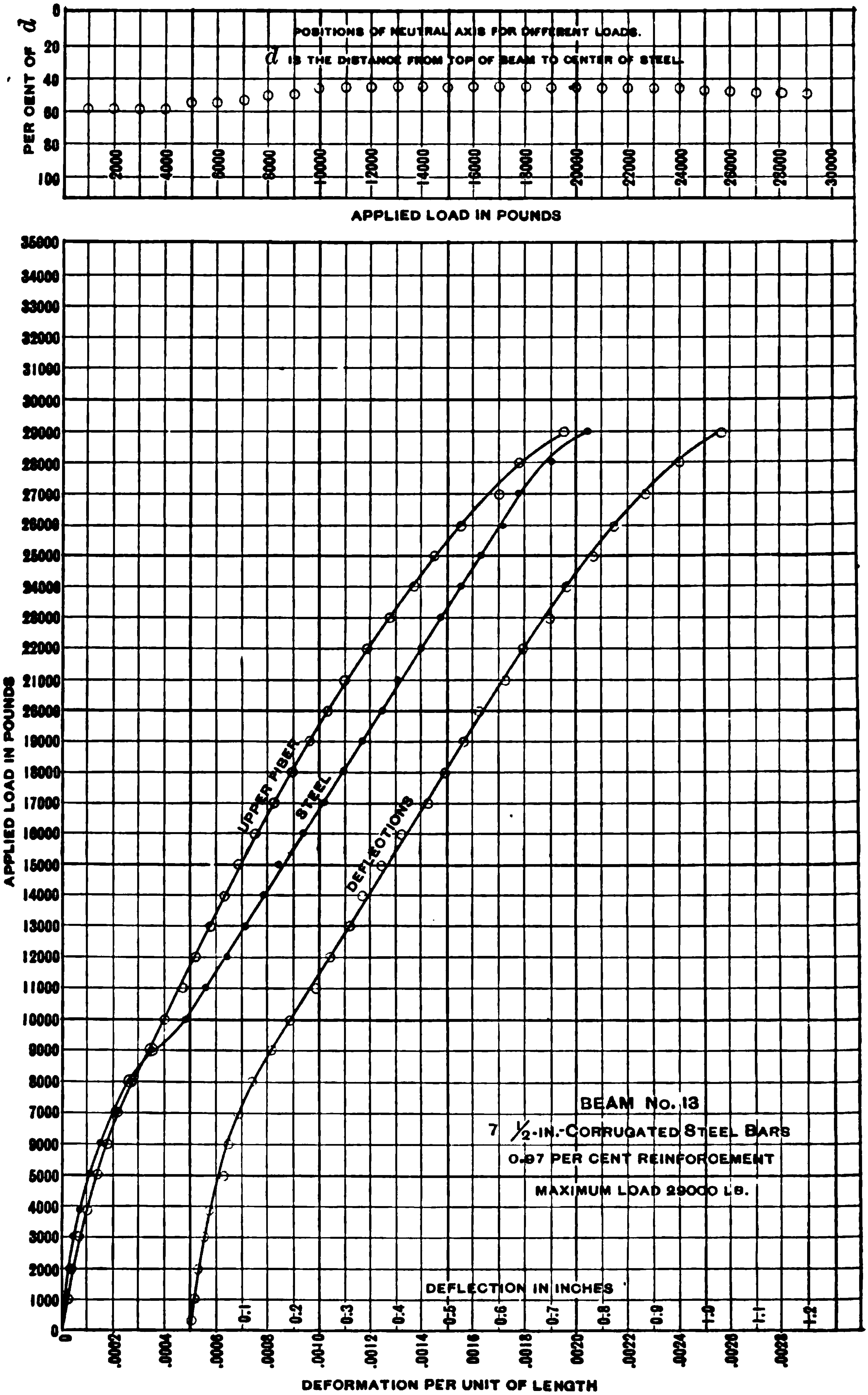

FIG. 89. — Typical Deformation and Deflection Curves of a Reinforced Beam. By Prof. A. N. Talbot. (*See p.* 287.)

seem probable that at these cracks the stress in the steel is more than is indicated by the average deformation for the full gage length.

Prof. Talbot states that at the load when the curve changes character, — which in the beam shown in the diagram is about 8 000 pounds total load, — there are probably invisible cracks in the lower portion of the beam. This change in direction of the curve, indicating a suddenly increased load upon the steel, is strong proof of the loss in tensional resistance of the concrete. Prof. Turneaure, moreover, in his experiments, at loads somewhat beyond the point of change in direction, actually discovered these minute cracks. He tested his beams upside down, that is, the load was applied upward, and the minute cracks or water-marks were shown by hair lines on the wet surface of the concrete. Prof. Turneaure* says:

> It has been found that by testing the beams when somewhat moist, a crack is made visible when exceedingly small, it appearing first as a narrow, wet streak perhaps ⅛-inch wide and a little later as a dark hair-like crack. It was not necessary to search for the lines with a microscope as under these conditions they were readily found.
>
> That the wet streak, called a "water-mark" hereafter, shows the presence of an actual crack was demonstrated last year by sawing out a strip of the concrete containing such a water-mark; the strip fell apart at the water-mark.
>
> In the plain concrete no water-marks or cracks were observed before rupture. Comparing the observed and calculated elongations of the reinforced concrete with those for the plain concrete at rupture, it will be seen that the initial cracking in the former occurs at an elongation practically the same as in the latter.
>
> The significance of these minute cracks is an open question. It has been supposed that concrete reinforced by steel will elongate about ten times as much before rupture as will plain concrete. These experiments show very clearly that rupture begins at about the same elongation in both cases. In the plain concrete total failure ensues at once; in the reinforced concrete rupture occurs gradually, and many small cracks may develop so that the total elongation at final rupture will be greater than in the plain concrete. In other words, the steel develops the full extensibility of a non-homogeneous material that otherwise would have an extension corresponding to the weakest section.

These results are somewhat at variance with the conclusions reached by Mr. Considère† in France. He was not able to locate these fine cracks, and therefore concluded that while the stretch of plain concrete was about 0.0001 of its length or about 0.01%, in combination with steel it could

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*Proceedings American Society for Testing Materials, 1904.

†Considère's Reinforced Concrete, p. 35.

actually attain a stretch twenty times this, or 0.2%. Because of this apparent action of the concrete, Mr. Considère in his formula for beams assumes the concrete to resist a certain amount of tension.

The stretch, or deformation, in the concrete of a reinforced beam may be approximately estimated from the pull, or stress, upon the steel and the modulus of elasticity of the latter, since

$$\text{elastic deformation} = \frac{\text{stress}}{\text{modulus of elasticity}}$$

For example, if the steel is pulled to 16 000 pounds per square inch, the stretch (disregarding initial tension) is $\frac{16\,000}{30\,000\,000} = 0.00053$.

Tensile Resistance in the Concrete. Professors Talbot and Turneaure both conclude that the tensile strength of concrete may be disregarded in the consideration of the ultimate load carried by a beam. This not only agrees with current practice in design but is in accordance with the conclusions of many European scientists* and with the Prussian regulations† issued by the Minister of Public Works in 1904. The tensile resistance of the concrete affects the deformation and deflection of the beam under the smaller loads, but if, as is customary, the working strength is taken as a definite fraction of the resistance at the elastic limit of the steel, *the tensile resistance of the concrete need not be considered in the design of reinforced beams.*

Prof. Turneaure says:

> The presence of the cracks of course seriously affects the tensile strength of the concrete, and, as they appear at an elongation corresponding to a stress in the steel of 5 000 pounds per square inch or less, it would seem that no allowance should be made for the tensile resistance of the concrete. Furthermore, if such cracks are present the calculation of the tensile resistance of reinforced concrete by the method used by Considère leads to no useful result. In his tests Considère determines the stress in the steel from measurements of its elongation and then assumes the concrete to carry the remainder of the load. Assuming the value of E to be uninfluenced by the concrete, this would be correct so long as the stress in the steel and in the concrete is uniform between points of measurement. As stated by Considère himself, such results are only average values. But the concrete may be cracked entirely through and yet possess a very considerable average tensile strength over a length of several inches. Obviously in that case an average is of no value; the strength of the concrete is really zero.

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*See Christophe's Béton Armé, pp. 516 to 535.

†*Engineering Record*, July 2, 1904, p. 25.

In practical design the most important question which arises is how far a concrete may be cracked without exposing the steel to corrosive influences. In this respect it seems to the writer that the minute cracks which appear in the early states of the tests can have very little influence. However, the entire question of the effect of cracks and pores in the concrete on the corrosion of the steel needs careful investigation.

LOCATION OF NEUTRAL AXIS

The location of the neutral axis after the load has been transferred to the steel, is given in formula (6) on page 298 and numerical values for different moduli of elasticity on page 285. As is evident from that formula, it varies with the strength and elasticity of both the concrete and steel. Because of the peculiar action of the deformations, as illustrated in Fig. 89, page 288, the location of the neutral axis changes as the load is applied.

An empirical formula suggested by Prof. Talbot for the location of the neutral axis under normal loading, is given on page 328.

Prof. Turneaure states with reference to his own tests and diagrams:

> The diagrams show the neutral axis to lie at first very near the center of the concrete beam. As the cracks develop it moves gradually nearer to the compression side. It should be noted that the neutral axis as here found, by measuring deformations over a length of 14 inches, is the average neutral axis over this distance. At the point where a crack exists, the elongation per inch of the steel is more than elsewhere and the neutral axis is therefore nearer the compression side than in the average position.

QUALITY OF REINFORCING STEEL

It is generally recognized that in beam design the yield point of the steel shall be considered as the point of failure of this material in a reinforced beam. Tests show that when the metal reaches its yield point, the beam sags, and this deflection, due to the stretch of the steel, and in some cases to the slipping of the steel because of its reduced cross-section, is likely to produce crushing in the concrete.

The yield point of ordinary mild steel purchased in the open market, as determined by the drop of the beam in testing (the true elastic limit is several thousand pounds lower), cannot safely be fixed at a higher value than 30 000 pounds per square inch, although frequently, and in fact in the majority of cases, a value of at least 36 000 pounds, and in many cases 40 000 pounds, will be found.

High steel, that is, steel containing a high percentage of carbon, has a much higher yield point than mild steel. If of first-class quality,* a mini-

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*See Specifications for First-class Steel, p. 38.

mum yield point may be placed at 50 000 or 55 000 pounds per square inch and much of it will reach 60 000 pounds. The ultimate strength should be not less than 105 000 pounds per square inch. Thus, if it can be safely employed in reinforced concrete, it is adapted to carry much higher stress than mild steel, and, conversely, a smaller percentage of it is required for the same moment of resistance. Many engineers do not approve of the use of high steel because of its brittleness, when of poor quality, and the danger of sudden accident, and because of the fact that it is prohibited in ordinary structural steel work.

Mild steel, that is, ordinary market steel, is manufactured and sold under such standard conditions that it may be safely used without test. High steel, on the other hand, must be very thoroughly tested. When tested, however, as per our specifications, page 38, it is entirely safe and to be preferred to mild steel. The objection to it for reinforced concrete is based largely upon the use of a poor quality of material. Another objection which has been raised is that before the elastic limit is reached, the stretch in the high steel may produce an excessive cracking in the concrete in the lower portion of the beam, and thus expose the steel to corrosion. The mere fact that cracks are visible does not prove that they are dangerous, because the steel is always designed to take the whole of the tension. This point remains to be definitely settled, but Mr. Considère's and Professors Talbot's and Turneaure's tests indicate that there is no dangerous cracking even with high steel until the yield point of the steel is reached. This fact can be positively determined by cutting sections from reinforced concrete beams which have been strained nearly to the elastic limit, and testing them for corrosion by the methods employed by Prof. Charles L. Norton. (See p. 427.) A yield point in steel of 30 000 pounds per square inch corresponds to a stretch of 0.0010 of its length and a yield point of 50 000 to a stretch of 0.00167. (See p. 290.)

A steel with a high modulus of elasticity would be particularly serviceable for reinforced concrete, because the higher the modulus of elasticity of a material, the less is the deformation under any given loading. Unfortunately, however, a high carbon steel has substantially the same modulus of elasticity (30 000 000 lb. per sq. in.) as ordinary merchant steel.

The brittleness feared in high steel is less dangerous in reinforced concrete than in many classes of structural steel work because the concrete protects it from shock, and also because smaller sections of steel are used in concrete beams than in steel beams, and the large and irregular shapes of the latter render them much more sensitive to irregular cooling during the process of their manufacture.

It may be stated, then, if the stretching of high steel when pulled to its allowable working stress is proved not to form dangerous cracks in the concrete, that high carbon steel, say, 0.56% to 0.60% carbon, of the quality used in the United States for making locomotive tires, is always better than mild steel for reinforced concrete provided the steel is well melted and rolled, and is comparatively free from impurities, such as phosphorus. However, a high carbon steel, unless limited by chemical analysis, and made under careful inspection, is in danger of being more brittle than low carbon steel. Its use, therefore, should be limited strictly to work important enough to warrant the ordering of a special steel and the taking of sufficient trouble on the part of the purchaser to insure strict adherence to the specification. Under such circumstances, the use of high steel is attended with much economy. In other words, since manufacturers cannot always be depended upon to exactly follow specifications of this nature, it is necessary that an inspector be sent to the works, or else that the steel be purchased from a reliable dealer who has had it thus carefully tested.

The specifications for first-class steel on page 38 are sufficiently explicit so that steel which comes up to them can be safely used. A steel which can be employed with safety for all the locomotive and car wheels of the country certainly cannot be discarded as unsafe for concrete, provided similar precautions are taken in its purchase.

FORMULAS FOR MOMENT OF RESISTANCE IN REINFORCED BEAMS*

For reasons discussed in the preceding paragraphs, the authors for the present have selected the straight line theory of distribution of stress with the concrete taking no tension, as that which best fits the most modern tests of reinforced beams.

This theory assumes the following hypotheses as a basis for practical design:

(1) A plane section before bending remains plane after bending.
(2) Tension is borne entirely by the steel.
(3) Initial tension or compression is absent in the steel.
(4) Adhesion of concrete to steel is perfect.
(5) Modulus of elasticity of concrete within the usual limits of stress is a constant.

Although further experiments are required to prove the actual correctness

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*The authors are indebted to Prof. Frank P. McKibben for assistance in compiling these formulas.

of some of these assumptions, our reasons for selecting this theory may be briefly recapitulated as follows:

(*a*) Beams designed by it and properly built will be unquestionably safe.

(*b*) Fine cracks are formed in the tension portion of the beam at an early stage in the loading which actually destroy the tensile resistance of the concrete.

(*c*) The modulus of elasticity in many tests has been shown to be approximately a constant up to nearly the breaking point, and therefore, for the present, until its true curve has been determined, it may be assumed constant.

(*d*) This theory is the simplest, and the most easily understood.

(*e*) It has been adopted by Mr. Christophe,* by the Prussian Minister of Public Works, 1904, and by several other authorities.

(*f*) The results from it may be readily compared with other theories.

A theory which assumes a different distribution of pressure, and also one which assumes the concrete to take a portion of the tension, are given in Appendix II. In the determination of the pressure in the concrete during the period termed by Prof. Talbot the first stage (see p. 287), the tensile resistance in the concrete must be taken into account. It is believed, however, that the most practical method for calculating beams is to disregard this tension and employ safe working stresses in the concrete and the steel.

The formulas for reinforced beams having steel in the upper portion to assist in resisting the compression are given in Appendix II. The T-section is likewise discussed there.

Turning now to the derivation of the formulas, we may represent the stresses in the beam by the diagram shown in Fig. 90, page 296. At any vertical section through the beam the concrete in the upper portion resists the forces which tend to compress it, and the steel in the lower part of the beam resists the forces which tend to stretch and break it in tension. The compressive resistance acts in one direction and the tensile resistance in another direction, as designated by the large arrows in the diagram. The center of tension in the steel is at the center of the rod, or, if there is more than one tier of rods, through the center of gravity of the set of rods. The center of pressure of the concrete passes through the center of gravity of the triangle which represents the compressive stresses. The reason for the assumption of the uniformly increasing pressure from the neutral axis to the outside fiber is discussed on page 286.†

Notation. — The symbols selected for the notation, to aid the memory,

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*Christophe's Béton Armé, 1902, p. 535.

†If the stress is assumed to vary as a parabola, the assumed pressure in the concrete will be much lower for the same moment of resistance, but if the strength of the beam is based upon the steel, similar percentages of steel will give nearly identical moments of resistance. (See p. 300.)

are as far as possible the initial letters of the words which they represent.

The base, C, of the triangle represents the maximum unit compression in the concrete, that is, the highest compressive stress in any portion of the beam, under any particular loading. It is evident that the portion of the beam which is subject to the greatest compression is its upper surface.

The unit tension, or pull, in the steel for any loading is designated by S.

By calling xd the distance from the outside compressive surface to the neutral axis, x becomes a constant ratio of this distance to the distance, d, from outside compression surface to center of steel, for beams of all dimensions composed of materials having like moduli of elasticity and containing the same percentage of steel. For a beam in which the depth to center steel, d, is unity, the distance to the neutral axis becomes x. Similarly, p is taken as the ratio of the cross-section of the steel to the area of the concrete above the steel. This is rational, since the extra concrete below the steel does not add to the strength of the beam. It should be noticed that p is taken as a ratio and not as a percentage; thus 1% steel is represented by $p = 0.01$.

Let

h = height of beam.

b = breadth of beam.

p = ratio of cross-section of steel to cross-section of beam above the center of gravity of the steel.

C = unit pressure in outside fiber of concrete.

S = unit pull, or tension, in steel.

E_c = modulus of elasticity of concrete in compression.

E_s = modulus of elasticity of steel.

$r = \frac{E_s}{E_c}$ = ratio of moduli of elasticity of steel to concrete.

d = distance from outside compressive fiber to center of gravity of steel.

xd = distance from outside compressive surface to neutral axis, in a beam having steel at depth, d, below the outside compressive surface.

x = ratio of depth of neutral axis to depth, d, of steel.

M_R = moment of resistance.

M_B = bending moment.

K = constant for a given steel and a given concrete (see p. 299).

e = extra concrete, *i.e.*, depth of concrete below center of gravity of steel.

F = factor of safety.

In substituting numerical values for the notation, the English system of weights and measures is employed:

Linear dimensions in inches (except length of span in feet).

Surface dimensions in square inches.

Stress, *i.e.*, either compression or pull, in pounds per square inch.

Deformation in inches.

Modulus of elasticity in pounds per square inch.

Moments in inch-pounds.

Formulas. Referring now to Fig. 90, there is formed by the internal resisting forces a couple. One arm represents the total compression in the concrete acting through its center of compression, viz., the center of gravity of the triangular area representing the compression and having for its base C and its height xd. The other arm represents the total tension in the steel acting through the center of gravity of the rod or group of rods.

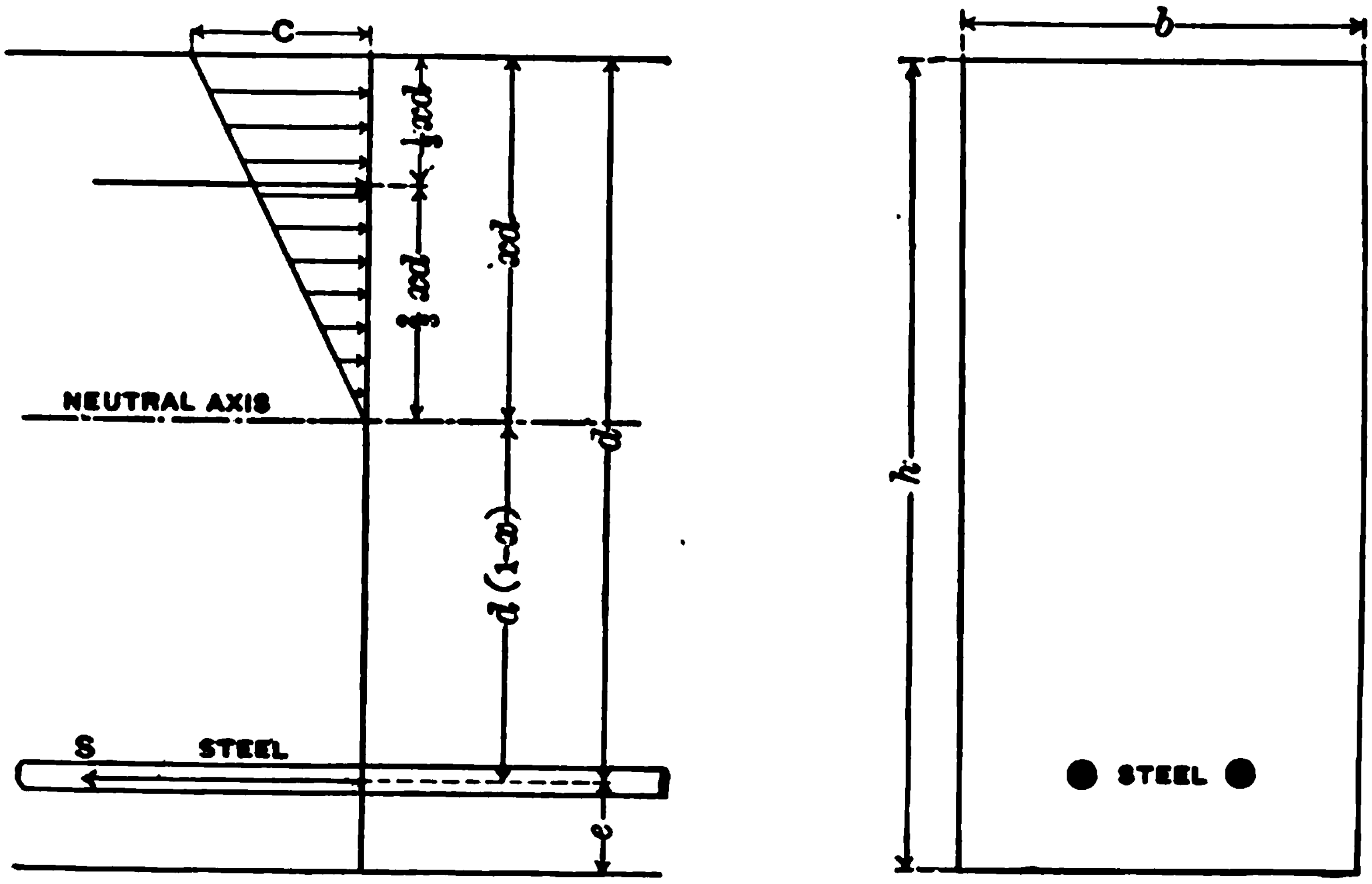

FIG. 90.—Resisting Forces in a Reinforced Concrete Beam. (*See p.* 296.)

For equilibrium in the beam, these two forces, parallel to each other and acting in opposite directions, must be equal. In other words, the total compression in the concrete must equal the total tension in the steel. If either one exceeds its maximum strength, the beam fails. These conditions are assumed to be true only after the point of loading is reached at which the tension is transferred to the steel, as otherwise the tension would be made up of two forces, the tension in the steel and the tension in the concrete, as discussed in Appendix II.

The moment of resistance of the couple must be equal to or greater than the bending moment produced by the live and dead loads.

Since it is assumed that a plane section before bending remains a plane section after bending, we have the proportion

$$\frac{\text{deformation in steel}}{\text{deformation in outside compressive concrete fibers}} = \frac{d\,(1-x)}{xd}$$

And since deformation $= \dfrac{\text{stress per square inch}}{\text{modulus of elasticity}}$ we have

$$\frac{\dfrac{S}{E_s}}{\dfrac{C}{E_c}} = \frac{d\,(1-x)}{xd} \quad \text{or} \quad \frac{S}{Cr} = \frac{1-x}{x} \tag{1}$$

From which

$$x = \frac{1}{1 + \dfrac{S}{Cr}} \tag{2}$$

Solving formula (1) for C

$$C = S\,\frac{x}{r\,(1-x)} \tag{3}$$

Now, as stated above, for equilibrium the total tension in the steel must be equal and opposite to the total compression in the concrete. The total tension in the steel is its unit tension, S, multiplied by the area of the steel, pdb, and the total compression in the concrete is represented by the area of the pressure triangle, $\frac{1}{2}C\,(xd)$ times the breadth of the beam, b. Equating these two stresses and cancelling out the db which occurs in both,

$$pS = \frac{Cx}{2} \tag{4}$$

If the value of x in formula (2) be substituted for the x in formula (4), we have

$$p = \frac{1}{2\left(\dfrac{S}{C}\right)\left(1 + \dfrac{S}{Cr}\right)} \tag{5}$$

For any given percentage of steel the values of S and C cannot be assumed independently, as they bear a constant ratio to each other.

Substituting the value of C in formula (3) for C in formula (4), we have

$$p = \frac{x}{2}\,\frac{x}{(1-x)\,r}$$

Solving this quadratic equation and adopting the positive sign before the square root,

$$x = -rp + \sqrt{2\,rp + (rp)^2}$$

or

$$x = rp\left(\sqrt{1 + \frac{2}{rp}} - 1\right) \tag{6}$$

We thus have x in terms of r and p, and from formula (6) the location of the neutral axis may be calculated with any percentage of steel for concrete and steel having known moduli of elasticity.

The moment of resistance is obtained from the couple by taking moments about the center of compression in the concrete, using for the force the total tension in the steel, which, as above, is $pSbd$, times the arm (see Fig. 90, p. 296), $d - \frac{xd}{3}$,

or

$$M_R = pSbd^2\left(1 - \frac{x}{3}\right) \tag{7}$$

The moment of resistance may also be expressed in terms of compression in the concrete by combining equations (4) and (7), or, more directly, by taking moments about the center of the tension in the steel, thus

$$M_R = \frac{Cxbd^2}{2}\left(1 - \frac{x}{3}\right) \tag{8}$$

Values for x with various percentages of steel and moduli of elasticity are given in the table on page 310.

The value of the moment of resistance, M, may also be expressed without using x by substituting in formulas (7) and (8) the value of p from formula (5) and the value of x from (2), thus giving

$$M_R = Sbd^2\left[\frac{1}{\frac{2S}{C}\left(1 + \frac{S}{Cr}\right)}\right]\left[1 - \frac{1}{3\left(1 + \frac{S}{Cr}\right)}\right] \tag{9}$$

or

$$M_R = \frac{Cbd^2}{2}\left[\frac{1}{1 + \frac{S}{Cr}}\right]\left[1 - \frac{1}{3\left(1 + \frac{S}{Cr}\right)}\right] \tag{10}$$

Formula (10) is apparently more complex than (7) and (8), but as the latter require the determination of x, formula (10) is more readily solved unless the table on page 310 is employed.

In the use of formula (10), S and C must be corresponding values and cannot be assumed independently of each other, since for any given percentage of steel the ratio of S to C is a constant. (See formula (5), p. 297.)

For concrete of a given quality, if we assume C to represent the unit breaking strength of concrete, — or rather the compression which we allow

the concrete to have at the time the steel reaches its elastic limit, — C is a constant, and E_c is also a constant. Similarly, for any steel, S and E_s are constants. Consequently, r, the ratio of E_s to E_c is also a constant, and if

F = required factor of safety,

we may let

$$K=\frac{S}{F}\left[\frac{1}{\frac{2S}{C}\left(1+\frac{S}{Cr}\right)}\right]\left[1-\frac{1}{3\left(1+\frac{S}{Cr}\right)}\right] \qquad (11)$$

Or in slightly different form,

$$K=\frac{C}{2F}\left[\frac{1}{1+\frac{S}{Cr}}\right]\left[1-\frac{1}{3\left(1+\frac{S}{Cr}\right)}\right] \qquad (12)$$

We may thus write in place of formula (10) the formula

$$M_R = Kbd^2 \qquad (13)$$

where K is a constant for any given concrete and steel.

Following directly from formula (13)

$$d=\sqrt{\frac{M_R}{Kb}}$$

Since in any beam the moment of resistance must be equal to or greater than the bending moment, we may substitute M_B for M_R. Also, since in Fig. 90, page 296, $d = h - e$, the formula may be written

$$h=\sqrt{\frac{M_B}{Kb}}+e \qquad (14)$$

from which the required height of the beam may be directly obtained.

If

W = the load in pounds upon a beam at its center (including one-half the weight of the beam).

w = load in pounds per running foot of the beam (including weight per foot of beam).

l = span in feet.

For a beam of height h_1, supported at both ends and loaded at the middle,

$$W=\frac{Kb(h_1-e)^2}{3\,l} \qquad (15)$$

$$h_1=\sqrt{\frac{3\,W\,l}{K\,b}}+e \qquad (16)$$

For a beam of height h_2, supported at both ends and uniformly loaded,

$$w = \frac{Kb\,(h_2 - e)^2}{1.5l^2} \tag{17}$$

$$h_2 = l\sqrt{\frac{1.5\,w}{K\,b}} + e \tag{18}$$

Values of *K* under various conditions and corresponding percentages of steel are given in the table on page 302. In this table the value of *K* corresponds to working (not to ultimate) loads, and therefore the values of *W* and *w* in formulas (15) to (18) must be working, or safe, loads.

COMPARISON OF STRAIGHT LINE AND PARABOLA THEORIES

Beams calculated by the straight line formulas, presented above, give higher apparent compression in the concrete for any given loading than if calculated with the assumption that the pressure in the concrete above the neutral axis varies as a curve. The lowest apparent pressure is obtained with the parabola theory of stress distribution, the formulas for which are given in Appendix II. Other theories lie between the parabola and the straight line.*

The moment of resistance for any beam, if it is based on the pull in the steel, is nearly identical by the two theories. If based on a certain pressure in the concrete, that is, if the steel is in excess, so that the pressure in the concrete is the limiting factor, the moment of resistance by the parabola theory is about 17% greater than by the straight line theory.

This apparent anomaly is explained by the fact that although the values of x for different percentages of steel and various moduli of elasticity of concrete, calculated by the parabola theory, are 10% to 12% smaller, — *i.e.*, the neutral axis is nearer the compressed surface by these per cents, — than when calculated by the straight line theory, the location of the center of pressure in the two cases is nearly the same, in fact, within less than 1%, and hence the arm of the couple, that is, the distance from the center of pressure to the center of pull, is practically the same in the two cases.

With similar loading, the nominal pressure in the concrete by the parabola theory averages about 15% less than when calculated by the straight line theory. If, for example, 625 pounds per square inch is assumed as safe compression by the straight line theory, 530 pounds per square inch should be used to obtain the same moment of resistance by the parabola theory.

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*See Discussion of Sanford E. Thompson to Prof. Talbot's paper in Proceedings American Society for Testing Materials, 1904.

TABLES OF DIMENSIONS OF BEAMS AND THEIR REINFORCEMENT

Tables are presented (1) for the purpose of furnishing a convenient means of determining the correct dimensions and reinforcement for beams under various loads, and (2) for comparing the effect upon the moment of resistance, of the assumption of different moduli of elasticity and of various allowable stresses in the concrete and steel, and of different percentages of reinforcement.

General tables adapted to all classes of beams, with values to be used in computation where special qualities of concrete and steel are known or assumed, so that the safe unit stresses may be adapted to suit particular conditions, are presented on pages 302 and 309. The table on page 313 is based upon certain assumptions which satisfy average conditions and give directly the safe loads for beams of various spans and depths with about 0.7% reinforcement, that is, a ratio of area of steel to cross-section of beam above steel of 0.0069. Tables for safe loading for slabs with various percentages of reinforcement are presented on pages 317 and 318.

All these tables have been calculated by the authors on the theory presented in the preceding pages.

Vertical or inclined reinforcement is treated on page 320.

Table for Use in Calculating Moments of Resistance and Dimensions of Beams. In the table on page 302 various values of K (see p. 299) are worked out for different assumed safe loads for concrete in compression and for steel in tension, and the percentage of steel corresponding to these loads is given. Theoretically, it is not strictly correct to apply working or safe loads upon the steel and concrete to the formulas presented on the preceding pages, because these formulas assume no tension in the concrete, and therefore can only truly apply after the tension in the concrete has been transferred, on account of cracks, to the steel. Practically, however, the same results are reached as though more nearly ultimate loads were employed, since the formula assumes a constant relation between the compression in the concrete and the tension in the steel. For the reason stated, the actual pull in the steel for any given compression in the concrete may be in reality less than the values in column (5), — that is, under working loads the concrete may actually carry a little pull, — but the ratio of this pull in the steel to its yield point is equivalent to the ratio of the given compression in the concrete to its compressive stress at the point when the steel reaches its yield point.

Instead of using the working load, as we have in working out the formula, it is often assumed that S represents the elastic limit or yield point of the

FOR USE WITH ORDINARY MILD STEEL

Data for determination of Moment of Resistance and Reinforcement.

RULE. — To find moment of resistance in any beam, multiply K times the breadth of the beam in inches by the square of the depth of steel, in inches, below upper surface: $M_R = Kbd^2$. (*See pages* 299 and 301.) Use items 48 to 53 for cinder concrete.

Item.	Modulus of Elasticity of Concrete. (E_c) lb. per sq. in.	Ratio of Moduli of Steel to Concrete. $\frac{E_s}{E_c} = r$	Working Strength of Concrete.* C lb. per sq. in.	Working Strength of Steel. S lb. per sq. in.	Ratio area of steel to beam above steel. p†	Safe working value of K *See page* 299.	Factor of Safety of Steel based on 60 000 lb. Ultimate Strength.	Factor of Safety of Concrete Based on Crushing Strength of 4000 lb. per sq. in.	3000 lb. per sq. in.	2500 lb. per sq. in.	2000 lb. per sq. in.	900 lb. per sq. in.	600 lb. per sq. in.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
1	4 000 000	7.5	750	16 000	0.0061	90	3.8	5.3	4.0	3.3	2.7		
2	4 000 000	7.5	700	16 000	0.0054	79	3.8	5.7	4.3	3.6	2.9		
3	4 000 000	7.5	625	16 000	0.0044	65	3.8	6.4	4.8	4.0	3.2		
4	4 000 000	7.5	750	14 000	0.0077	97	4.3	5.3	4.0	3.3	2.7		
5	4 000 000	7.5	700	14 000	0.0068	87	4.3	5.7	4.3	3.6	2.9		
6	4 000 000	7.5	625	14 000	0.0056	72	4.3	6.4	4.8	4.0	3.2		
7	4 000 000	7.5	750	12 000	0.0100	107	5.0	5.3	4.0	3.3	2.7		
8	4 000 000	7.5	700	12 000	0.0089	96	5.0	5.7	4.3	3.6	2.9		
9	4 000 000	7.5	625	12 000	0.0073	80	5.0	6.4	4.8	4.0	3.2		
10	4 000 000	7.5	550	12 000	0.0059	64	5.0	7.3	5.4	4.6	3.6		
11	3 000 000	10	700	16 000	0.0067	96	3.8	5.7	4.3	3.6	2.9		
12	3 000 000	10	625	16 000	0.0055	80	3.8	6.4	4.8	4.0	3.2		
13	3 000 000	10	550	16 000	0.0044	64	3.8	7.3	5.4	4.6	3.6		
14	3 000 000	10	700	14 000	0.0083	104	4.3	5.7	4.3	3.6	2.9		
15	3 000 000	10	625	14 000	0.0069	87	4.3	6.4	4.8	4.0	3.2		
16	3 000 000	10	550	14 000	0.0055	70	4.3	7.3	5.4	4.6	3.6		
17	3 000 000	10	700	12 000	0.0108	113	5.0	5.7	4.3	3.6	2.9		
18	3 000 000	10	625	12 000	0.0089	95	5.0	6.4	4.8	4.0	3.2		
19	3 000 000	10	550	12 000	0.0072	77	5.0	7.3	5.4	4.6	3.6		
20	3 000 000	10	500	12 000	0.0061	66	5.0	8.0	6.0	5.0	4.0		
21	2 500 000	12	625	16 000	0.0062	89	3.8	6.4	4.8	4.0	3.2		
22	2 500 000	12	550	16 000	0.0050	73	3.8	7.3	5.4	4.6	3.6		
23	2 500 000	12	500	16 000	0.0043	62	3.8	8.0	6.0	5.0	4.0		
24	2 500 000	12	625	14 000	0.0078	96	4.3	6.4	4.8	4.0	3.2		
25	2 500 000	12	550	14 000	0.0063	79	4.3	7.3	5.4	4.6	3.6		
26	2 500 000	12	500	14 000	0.0054	68	4.3	8.0	6.0	5.0	4.0		
27	2 500 000	12	550	12 000	0.0081	86	5.0	7.3	5.4	4.6	3.6		
28	2 500 000	12	500	12 000	0.0069	74	5.0	8.0	6.0	5.0	4.0		
29	2 500 000	12	450	12 000	0.0058	63	5.0	8.9	6.7	5.6	4.4		
30	2 000 000	15	550	16 000	0.0058	83	3.8	7.3	5.4	4.6	3.6		
31	2 000 000	15	500	16 000	0.0050	71	3.8	8.0	6.0	5.0	4.0		
32	2 000 000	15	450	16 000	0.0042	60	3.8	8.9	6.7	5.6	4.4		
33	2 000 000	15	550	14 000	0.0073	89	4.3	7.3	5.4	4.6	3.6		
34	2 000 000	15	500	14 000	0.0062	77	4.3	8.0	6.0	5.0	4.0		
35	2 000 000	15	450	14 000	0.0052	65	4.3	8.9	6.7	5.6	4.4		
36	2 000 000	15	500	12 000	0.0080	84	5.0	8.0	6.0	5.0	4.0		
37	2 000 000	15	450	12 000	0.0068	71	5.0	8.9	6.7	5.6	4.4		
38	2 000 000	15	400	12 000	0.0056	59	5.0	10.0	7.5	6.2	5.0		
39	1 500 000	20	500	16 000	0.0060	84	3.8	8.0	6.0	5.0	4.0		
40	1 500 000	20	450	16 000	0.0051	71	3.8	8.9	6.7	5.6	4.4		
41	1 500 000	20	400	16 000	0.0042	59	3.8	10.0	7.5	6.2	5.0		
42	1 500 000	20	500	14 000	0.0074	90	4.3	8.0	6.0	5.0	4.0		
43	1 500 000	20	450	14 000	0.0063	77	4.3	8.9	6.7	5.6	4.4		
44	1 500 000	20	400	14 000	0.0052	64	4.3	10.0	7.5	6.2	5.0		
45	1 500 000	20	450	12 000	0.0080	83	5.0	8.9	6.7	5.6	4.4		
46	1 500 000	20	400	12 000	0.0067	69	5.0	10.0	7.5	6.2	5.0		
47	1 500 000	20	350	12 000	0.0054	56	5.0	11.4	8.6	7.1	5.7		
48	850 000	35	225	16 000	0.0023	33	3.8					4.0	3.0
49	850 000	35	150	16 000	0.0012	17	3.8					6.0	4.0
50	850 000	35	225	14 000	0.0029	36	4.3					4.0	3.0
51	850 000	35	150	14 000	0.0015	19	4.3					6.0	4.0
52	850 000	35	225	12 000	0.0037	39	5.0					4.0	3.0
53	850 000	35	150	12 000	0.0019	20	5.0					6.0	4.0

*If the parabola theory of stress distribution is employed, the assumed working strength in the concrete, — with the same steel — will be about 15% lower than the figures in column (4). (*See p.* 300.)

†For percentages of steel multiply values in column (6) by 100.

steel and *C* the ultimate compressive strength of the concrete. The latter would be the more rational method if the ratio of the yield point of steel to its working strength could be assumed to be equal to the factor of safety for concrete. In the present stage of the science of reinforced concrete design, however, this is not permissible. To illustrate, the yield point of ordinary mild steel — although it often reaches 36 000 pounds or even more than this — cannot be counted upon for more than 30 000 pounds per square inch. The average crushing strength of 1:2:4 concrete, at the age of one month, may be taken at 2 440 pounds per square inch (see p. 242). Now, a ratio of yield point to working strength of 2 for the steel (nearly corresponding to a factor of safety of 4 on its ultimate strength) gives 15 000 pounds, a common value for safe tensile stress. But this ratio of 2 applied as a factor of safety to the concrete gives 1220 pounds, a dangerous extreme. If, on the other hand, a factor of safety of 4 were selected, so as to reduce the safe load upon the concrete to 610 pounds, the stress in the steel would be only 7 500 pounds, and consequently a much larger percentage of steel would be used than is necessary. The inexperienced designer is warned against falling into this error of thus using a higher percentage of steel than is necessary.

To avoid this high compression in the concrete, or, on the other hand, the low tension in the steel, a stress in the concrete of, say, 1220 pounds might be selected to correspond to the 30 000 pounds in the steel. It is simpler, however, and, as has been stated, amounts to the same thing, to divide these stresses by 2 before making the calculation, and use, for example, the safe stress of 610 per square inch in compression for the 1:2:4 concrete and 15 000 pounds per square inch in tension for the steel.

In the method adopted, that is, starting with working loads, the value of *K* (see p. 299) and the moment calculated from it are safe working values based upon the factor for safety of either the concrete or steel, whichever is selected as the limiting material.

Other factors of safety than those assumed in calculating the table may be adopted for concrete of any ultimate strength by multiplying any value of *K* corresponding to the required percentage of steel and the elasticity of the concrete by one of the corresponding factors of safety and dividing the product by the desired factor. However, *the safe working load of neither the concrete nor the steel must be exceeded.*

The working strengths of the concrete given in column (4) appear high, but by comparing the factors of safety corresponding to various ultimate strengths, as given in columns (9) to (14) with the average values of the strength of concrete at one and six months in the table on page 242, and

also with the results of actual tests, also given in Chapter XIII, it is evident that ample allowance has been made for ordinary conditions. If a larger or smaller factor of safety is required, the value of *K* can be readily calcuated or obtained by interpolation or exterpolation.

Various combinations of the modulus of elasticity, compressive strength of concrete, and tensile strength of steel are presented, so that the calculator may select those which correspond to the materials he is to use and which are adapted to the character of the structure. In cases where the work is not of sufficient importance to warrant making special tests or examinations of the materials, the tables on pages 313, 317, and 318 may be employed directly. The method of using the general table on page 302 is illustrated in examples 1 to 9.

EXAMPLES OF BEAM AND SLAB DESIGN

The following examples illustrate the methods of computing moments of resistance and dimensions of simple beams and slabs.

Examples 1 to 9 are from the general table on page 302, and examples 10 to 14 illustrate the use of the tables on pages 313 and 317 for beams and slabs.

If no proportions of concrete are given, 1: 2½: 5 are assumed.

The table of areas and weights of rods on page 311 will be found convenient for converting areas of steel into mercantile standards.

The weights of reinforced concrete beams and slabs may be readily determined from column (22), page 313, and column (15), page 317.

Example 1. What is the moment of resistance of a beam of reinforced 1: 2½: 5 concrete of 20 feet span, 8 inches wide, and 17 inches deep, with one 15/16-inch square rod imbedded 2 inches above the bottom so as to give an effective depth of 15 inches, the pressure in the concrete being limited to 625 pounds per square inch, and the pull in the steel to 14 000 pounds per square inch?

Solution. A 15/16-inch rod has a section of 0.88 inches, corresponding in the given beam to 0.73% steel. Assuming a modulus of elasticity of concrete of 3 000 000, we find at once that the conditions are nearly satisfied by item (15) in the table, page 302, lying between this item and item (18). Interpolating, to provide for the slight excess of steel over item (15) gives 88 for the value of *K*. Introducing this value of *K* together with the known values for *h*, *b* and *e* into the formula for M_R at the top of the table, the moment of resistance is found to be $88 \times 8 \times 15^2 = 158\,400$ inch-pounds. In determining the load which this beam will carry, the weight of the beam must be included in the load.

Example 2. With the same concrete and steel as in Example 1, what are the required dimensions and reinforcement (with steel imbedded 2 inches) for a series of beams of 12-foot span, spaced 10 feet apart, and supporting a floor loaded with 100 pounds per square foot including the weight of the floor?

Solution. The loading upon the beam per linear foot (not including the weight of the beam) is 1 000 pounds. If the effect of the T-section formed by the combined beam and slab is disregarded, we may select at once item (15) from the table, which gives us with the allowed pull on the steel a ratio of steel of 0.0069 (0.69%), and a value of K of 87. For trial, assume a breadth of beam of 12 inches, and a weight of beam per running foot of 250 pounds, which added to the 1 000 pounds gives a total weight per running foot of 1 250 pounds. Substituting these values in formula (18) on page 300, assuming $e = 2$, we find directly that the depth of the beam, h, $= 12\sqrt{\frac{1.5 \times 1250}{87 \times 12}} + 2 = 18.1$ inches, and the cross-section of steel $(18.1 - 2.0) \times 12 \times 0.0069 = 1.33$ square inches, for which may be used two $\frac{15}{16}$-inch round rods or two $\frac{13}{16}$-inch square rods.

If the height in any case is considered out of proportion to the assumed breadth, or if the weight of the beam is much different from that assumed, a new calculation must be made.

Example 3. What will be the effect of selecting a larger percentage, say 0.9%, of steel, for the beam in Example 2?

Solution. From the table on page 302 we find that item (18) satisfies the conditions by giving a steel ratio of 0.0089, corresponding to 0.9%. The value of K is therefore 95 instead of 87. Substituting in formula (18) as before, we find the total depth of the beam must be $12\sqrt{\frac{1.5 \times 1250}{95 \times 12}} + 2 = 17.4$ inches, showing that a depth of only 0.7 inch in thickness of concrete is saved by thus increasing the steel.

Example 4. What would be the moment of resistance of the beam in Example 1 if the crushing strength of the concrete be limited to 500 pounds per square inch?

Solution. From inspection of the table it is evident that the percentage of steel is much larger than should be employed, because reducing the allowable stress in the concrete proportionally reduces the working stress in the steel, if the percentage of steel remains the same. From formulas (11), (12) and (13), page 299, it is evident, that with the same percentage of steel, the value of K and also the value of M_R are proportional to the allow-

able stress in the concrete or the steel, whichever is the limiting factor. Consequently, the moment of resistance will be $158\ 400 \times \frac{500}{625} = 126\ 700$.

Example 5. What is the value of K and the ratio of steel if pressure in concrete is limited to 400 pounds per square inch and pull in steel to 12 000 pounds per square inch, the modulus of elasticity of concrete being assumed at 3 000 000 pounds per square inch?

Solution. Approximate values, which are sufficiently exact, may be obtained from the table, page 302, by exterpolation below item (20), from which K equals 44, and ratio of steel, p, = 0.0039.

Example 6. What is the value of K for a beam in which the pressure in the concrete is 625 pounds per square inch, the pull in the steel 14 000 pounds, and the area of steel 0.9%, the modulus of elasticity of the concrete being 3 000 000?

Solution. The requirements in the example are impossible. With the pressure in the concrete limited to 625 pounds per square inch, the pull in the steel, if 0.9% is used, cannot be as high as 14 000 pounds. From item (18), page 302, in which the ratio of steel is 0.0089 (0.89%), $K = 95$. The pull in the steel is 12 000 pounds. Comparing this item with item (15) in the same table, it is evident that an increase of 29% in the area of the steel, *i.e.*, from ratio 0.0069 to ratio 0.0089, increases the value of K, and therefore the moment of resistance, scarcely 10%.

Example 7. In item (15), page 302, what are the factors of safety of the steel and of 1 : 2½ : 5 concrete at the age of 6 months?

Solution. The factor of safety of the steel is obtained directly from Column (8) as 4.3. From the table on page 242, the average compressive strength of 1 : 2½ : 5 concrete, at the age of 6 months, is 2940 pounds per square inch, hence from column (10), page 302, the factor of safety corresponding to item (15) is 4.8.

Example 8. What is the moment of resistance and what rods may be economically used in a beam of cinder concrete 12 inches wide by 18 inches deep, if the pull in the steel is limited to 14 000 per square inch, and the pressure in the concrete to 225 pounds per square inch, assuming a modulus of elasticity for cinder concrete of 850 000?

Solution. Item (50), page 302, satisfies the conditions. Therefore, $K = 36$ and $p = 0.0029$. Assuming the steel to be 2 inches above the bottom, the depth of steel, d, is 16. $M_R = Kbd^2 = 36 \times 12 \times 16^2 = 110\ 592$ inch-pounds. Area steel is $12 \times 16 \times 0.0029 = 0.56$ square inches, which from table on page 321 calls for one ¾-inch diameter round rod.

A comparison of this example with Example 2 will show the poor economy of employing cinder concrete for beams.

Example 9. What would be the value of K and the percentage of steel in a beam of reinforced broken stone concrete if calculated by the parabola theory of pressure distribution, if the working pressure in the concrete is 550 pounds, the allowable pull in the steel 16 000, and the modulus of elasticity of the concrete, 4 000 000?

Solution. From page 302, since under similar conditions the apparent pressure in the concrete is 15% less when calculated by the parabola than by the straight line theory, 550 pounds by the parabola theory is equivalent to $\frac{550}{0.85} = 650$ pounds per square inch by the straight line theory. Interpolating between items (2) and (3) gives $K = 69$, and $p = 0.0047$, corresponding to 0.47% steel.

Example 10. What safe load per square foot can be supported by a slab 5 inches thick and 10-foot span reinforced with ½-inch round bars placed 8 inches apart?

Solution. From slab table, page 317, since the given reinforcement from page 311 is equivalent to $0.196 \times 1\frac{1}{2} = 0.294$ square inches for one foot of width, we find by inspection that for a 5-inch slab the nearest area of steel in column (18) is 0.288. Hence, the total safe load for a 10-foot span is slightly more than 121 pounds, say, 123 pounds per square foot; and deducting the weight per square foot of the slab, column (15) gives $123 - 64 = 59$ pounds per square foot safe live load.

Example 11. What safe load per square foot can be placed upon an 8-inch slab, 16-foot span, having steel reinforcement of 0.007?

Solution. Since by rule 3, on page 317, total loads are inversely proportional to the squares of the span, the load for a 16-foot slab is ¼ the load for an 8-foot slab. For the total safe load of an 8-foot slab, we must interpolate between steel ratios of 0.006 and 0.008, thus obtaining $\frac{581 + 699}{2} = 640$ pounds per square foot. For the 16-foot slab, the total safe load is therefore $\frac{640}{4} = 160$ pounds, and deducting the weight of the slab from column (15) gives a net live load of $160 - 103 = 57$ pounds per square foot.

Example 12. Using the table of beams, page 313, what should be dimensions and reinforcements for the beam described in Example 2?

Solution. The assumed stresses are the same as those adopted in the Beam Table. Making the same assumptions as in the solution to Example

2 gives a total load per inch of width of $\frac{1250}{12} = 104$ pounds running foot. Referring directly to the Beam Table, we find that the total depth corresponding to a 12-foot beam with this load is about 17½ inches, but as column (24) shows that the table assumes only 1½ inches of concrete below the steel, while the example assumes 2 inches, ½-inch must be added to the height of beam, giving 18 inches. The reinforcement from column (25) is $0.111 \times 12 = 1.33$ square inches.

Example 13. What total load per foot of length can be carried by a 12-foot beam 12 inches wide and 25 inches deep?

Solution. There is no value in the table for a beam whose total depth is 25 inches, but since, from rule 4, loads are proportional to the depth of the steel, we may calculate the load from the load for a 26-inch beam 12 inches wide. Assuming in both cases that the depth to steel, d, is 2 inches less than the total depth, we have $232 \times \frac{23^2}{24^2} \times 12 = 2560$ pounds per running foot of beam.

Example 14. What should be the dimensions and reinforcement for the beam described in Example 2 if 1:2:4 concrete is used, and the values are to be obtained from the beam table, page 313?

Solution. Since from rule 5, page 313, a beam of 1:2:4 concrete suitably reinforced will sustain about 20% greater load than 1:2½:5 concrete, we may assume the total load, instead of 1250 pounds per foot, to be $\frac{1250}{1.20} = 1042$ pounds per running foot, or 87 pounds per running foot per inch of width. This for a 12-foot span corresponds to a depth of beam of about 16½ inches, and since the steel must be increased by 20%, the area of reinforcement will be $0.101 \times 12 \times 1.20 = 1.46$ square inches.

GENERAL TABLE FOR HIGH CARBON STEEL

The table on page 309 gives values of K and also the required percentage of steel, with different stresses in the concrete and the steel, if first-class or high steel is used for the reinforcement. High steel is a most dangerous material to use unless it is thoroughly tested, and it must conform in mechanical strength and in chemical composition to our specifications on page 38, in which case it is entirely safe and is to be preferred to ordinary merchant steel.

Beams designed by this table are liable to show slight cracks on their

under surfaces before the working strength of the steel is reached, but such cracks are by no means dangerous to safety.

FOR USE ONLY WITH FIRST CLASS HIGH STEEL*

Data for determination of Moment of Resistance and Reinforcement.

RULE. — To find moment of resistance in any beam multiply K times the breadth of the beam, in inches, by the square of the depth of steel, in inches, below upper surface: $M_R = Kbd^2$. (*See pages* 299 and 301.)

Item.	Modulus of Elasticity of Concrete. (E_c) lb. per sq. in.	Ratio of Moduli of Steel to Concrete. $\frac{E_s}{E_c} = r$	Working Strength of Concrete.† C lb. per sq. in.	Working Strength of Steel. S lb. per sq. in.	Ratio area of steel to beam above steel. p‡	Safe working value of K (*See page* 299.)	Factor of Safety of Steel based on 100 000 lb. Ultimate Strength.	Factor of Safety of Concrete Based on Crushing Strength of			
								4000 lb. per sq. in.	3000 lb. per sq. in.	2500 lb. per sq. in.	2000 lb. per sq. in.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
1	4 000 000	7.5	750	25 000	0.0028	66	4.0	5.3	4.0	3.3	2.7
2	4 000 000	7.5	700	25 000	0.0024	57	4.0	5.7	4.3	3.6	2.9
3	4 000 000	7.5	625	25 000	0.0020	47	4.0	6.4	4.8	4.0	3.2
4	4 000 000	7.5	550	25 000	0.0016	37	4.0	7.3	5.4	4.6	3.6
5	4 000 000	7.5	750	20 000	0.0041	76	5.0	5.3	4.0	3.3	2.7
6	4 000 000	7.5	700	20 000	0.0036	68	5.0	5.7	4.3	3.6	2.9
7	4 000 000	7.5	625	20 000	0.0030	56	5.0	6.4	4.8	4.0	3.2
8	4 000 000	7.5	550	20 000	0.0024	44	5.0	7.3	5.4	4.6	3.6
9	3 000 000	10	700	25 000	0.0031	71	4.0	5.7	4.3	3.6	2.9
10	3 000 000	10	625	25 000	0.0025	58	4.0	6.4	4.8	4.0	3.2
11	3 000 000	10	550	25 000	0.0020	47	4.0	7.3	5.4	4.6	3.6
12	3 000 000	10	500	25 000	0.0017	39	4.0	8.0	6.0	5.0	4.0
13	3 000 000	10	700	20 000	0.0045	83	5.0	5.7	4.3	3.6	2.9
14	3 000 000	10	625	20 000	0.0037	69	5.0	6.4	4.8	4.0	3.2
15	3 000 000	10	550	20 000	0.0030	55	5.0	7.3	5.4	4.6	3.6
16	3 000 000	10	500	20 000	0.0025	47	5.0	8.0	6.0	5.0	4.0
17	2 500 000	12	625	25 000	0.0029	67	4.0	6.4	4.8	4.0	3.2
18	2 500 000	12	550	25 000	0.0023	53	4.0	7.3	5.4	4.6	3.6
19	2 500 000	12	500	25 000	0.0019	45	4.0	8.0	6.0	5.0	4.0
20	2 500 000	12	625	20 000	0.0043	78	5.0	6.4	4.8	4.0	3.2
21	2 500 000	12	550	20 000	0.0034	63	5.0	7.3	5.4	4.6	3.6
22	2 500 000	12	500	20 000	0.0029	53	5.0	8.0	6.0	5.6	4.0
23	2 000 000	15	550	25 000	0.0027	63	4.0	7.3	5.4	4.6	3.6
24	2 000 000	15	500	25 000	0.0023	53	4.0	8.0	6.0	5.0	4.0
25	2 000 000	15	450	25 000	0.0019	44	4.0	8.9	6.7	5.6	4.4
26	2 000 000	15	550	20 000	0.0040	72	5.0	7.3	5.4	4.6	3.6
27	2 000 000	15	500	20 000	0.0034	62	5.0	8.0	6.0	5.0	4.0
28	2 000 000	15	450	20 000	0.0028	52	5.0	8.9	6.7	5.6	4.4
29	1 500 000	20	500	25 000	0.0029	65	4.0	8.0	6.0	5.0	4.0
30	1 500 000	20	450	25 000	0.0024	54	4.0	8.9	6.7	5.6	4.4
31	1 500 000	20	400	25 000	0.0019	45	4.0	10.0	7.5	6.2	5.0
32	1 500 000	20	500	20 000	0.0042	74	5.0	8.0	6.0	5.0	4.0
33	1 500 000	20	450	20 000	0.0035	63	5.0	8.9	6.7	5.6	4.4
34	1 500 000	20	400	20 000	0.0029	52	5.0	10.0	7.5	6.2	5.0

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*High steel should never be used unless it will meet the specifications on page 38.

†If the parabola theory of stress distribution is employed, the assumed working strength in the concrete, — with the same steel — will be about 15% lower than the figures in column (4). (*See p.* 300.)

‡For percentages of steel, multiply values in column (6) by 100.

TABLE OF PROPORTIONAL DEPTHS OF NEUTRAL AXIS

The table below gives the proportional depths of the neutral axis calculated from formula (6) on page 298 for various percentages of steel and moduli of elasticity. Its use is *not* advised for ordinary calculations of moments of resistance and dimensions of beams or slabs, because it presents no means of determining, without further calculation, the stress in the steel or the concrete, and therefore is liable to lead to uneconomical design. Its principal use is for determining the moment of resistance, and consequently the safe loading for beams already built.

As stated in the rule at the foot of the table, the values are to be used in

Proportional Depth of Neutral Axis below top of Beam for different per cents of Steel and various assumptions of Elasticity. (See p. 310.)

p Ratio of area of steel to area of cross-section of beam above steel.	**x** Ratio of depth of neutral axis to depth of center of steel below most compressed surface of beam.								
	Ratios of Modulus of Elasticity of Steel to Modulus of Concrete in Compression, $\frac{E_s}{E_c} = r$								
	6	7.5	10	12	15	20	30	35	40
0.001	0.101	0.115	0.138	0.143	0.158	0.181	0.217	0.232	0.246
0.002	0.184	0.159	0.181	0.196	0.217	0.246	0.292	0.311	0.328
0.003	0.173	0.191	0.217	0.235	0.258	0.292	0.344	0.365	0.384
0.004	0.196	0.217	0.246	0.266	0.292	0.328	0.384	0.420	0.428
0.005	0.217	0.239	0.270	0.292	0.320	0.358	0.418	0.442	0.464
0.006	0.235	0.258	0.292	0.314	0.344	0.384	0.446	0.471	0.493
0.007	0.251	0.276	0.311	0.334	0.365	0.407	0.471	0.497	0.519
0.008	0.266	0.292	0.328	0.353	0.384	0.428	0.493	0.519	0.412
0.009	0.279	0.306	0.344	0.369	0.402	0.446	0.513	0.539	0.562
0.010	0.292	0.320	0.358	0.384	0.418	0.463	0.531	0.557	0.584
0.012	0.315	0.344	0.384	0.402	0.446	0.493	0.562	0.588	0.611
0.014	0.334	0.364	0.407	0.436	0.471	0.519	0.588	0.614	0.638
0.016	0.353	0.384	0.428	0.457	0.493	0.542	0.611	0.637	0.660
0.018	0.369	0.402	0.446	0.476	0.513	0.562	0.631	0.657	0.680
0.020	0.384	0.418	0.463	0.493	0.531	0.580	0.649	0.675	0.697
0.030	0.446	0.483	0.531	0.562	0.599	0.649			
0.040	0.493	0.531	0.580	0.611	0.649	0.697			
0.050	0.531	0.569	0.618	0.649	0.686	0.732			

NOTE: — This table is given for use in connection with formulas (7) and (8), page 298, for the study of beams already built.

RULE: — To find maximum safe (or ultimate) resisting moment, select maximum safe (or ultimate) values for S and C. Substitute them, and also the value of x from the above table, in formulas (7) and (8) (p. 298) and select the lower value of M_R.

connection with either formula (7) or formula (8) on page 298, whichever gives the lower result.

Areas and Weights of Square and Round Rods and Circumferences of Round Rods.
(*See p.* 312.)

One cubic foot weighs 490 lb.

Thickness or Diameter in inches.	Area of Square Rod in square inches.	Area of Round Rod in square inches.	Circumference of Round Rod in inches.	Weight of Square Rod One Foot Long.	Weight of Round Rod One Foot Long.	Thickness or Diameter in inches.	Area of Square Rod in square inches.	Area of Round Rod in square inches.	Circumference of Round Rod in inches.	Weight of Square Rod One Foot Long.	Weight of Round Rod One Foot Long.
0						2	4.0000	3.1416	6.2832	13.60	10.68
1/16	0.0039	0.0031	0.1963	0.013	0.010	1/16	4.2539	3.3410	6.4795	14.46	11.36
1/8	0.0156	0.0123	0.3927	0.053	0.042	1/8	4.5156	3.5466	6.6759	15.35	12.06
3/16	0.0352	0.0276	0.5890	0.119	0.094	3/16	4.7852	3.7583	6.8722	16.27	12.78
1/4	0.0625	0.0491	0.7854	0.212	0.167	1/4	5.0625	3.9761	7.0686	17.22	13.52
5/16	0.0977	0.0767	0.9817	0.333	0.261	5/16	5.3477	4.2000	7.2649	18.19	14.28
3/8	0.1406	0.1104	1.1781	0.478	0.375	3/8	5.6406	4.4301	7.4613	19.18	15.07
7/16	0.1914	0.1503	1.3744	0.651	0.511	7/16	5.9414	4.6664	7.6576	20.20	15.86
1/2	0.2500	0.1963	1.5708	0.850	0.667	1/2	6.2500	4.9087	7.8540	21.25	16.69
9/16	0.3164	0.2485	1.7671	1.076	0.845	9/16	6.5664	5.1572	8.0503	22.33	17.53
5/8	0.3906	0.3068	1.9635	1.328	1.043	5/8	6.8906	5.4119	8.2467	23.43	18.40
11/16	0.4727	0.3712	2.1598	1.608	1.262	11/16	7.2227	5.6727	8.4430	24.56	19.29
3/4	0.5625	0.4418	2.3562	1.913	1.502	3/4	7.5625	5.9396	8.6394	25.00	20.20
13/16	0.6602	0.5185	2.5525	2.245	1.763	13/16	7.9102	6.2126	8.8357	26.90	21.12
7/8	0.7656	0.6013	2.7489	2.603	2.044	7/8	8.2656	6.4918	9.0321	28.10	22.07
15/16	0.8789	0.6903	2.9452	2.989	2.347		8.6289	6.7771	9.2284	29.34	23.04
1	1.0000	0.7854	3.1416	3.400	2.670	3 15/16	9.0000	7.0686	9.4248	30.60	24.03
1/16	1.1289	0.8866	3.3379	3.838	3.014	1/16	9.3789	7.3662	9.6211	31.89	25.04
1/8	1.2656	0.9940	3.5343	4.303	3.379	1/8	9.7656	7.6699	9.8175	33.20	26.08
3/16	1.4102	1.1075	3.7306	4.795	3.766	3/16	10.160	7.9798	10.014	34.55	27.13
1/4	1.5625	1.2272	3.9270	5.312	4.173	1/4	10.563	8.2958	10.210	35.92	28.20
5/16	1.7227	1.3530	4.1233	5.857	4.600	5/16	10.973	8.6179	10.407	37.31	29.30
3/8	1.8906	1.4849	4.3197	6.428	5.049	3/8	11.391	8.9462	10.603	38.73	30.42
7/16	2.0664	1.6230	4.5160	7.026	5.518	7/16	11.816	9.2806	10.799	40.18	31.56
1/2	2.2500	1.7671	4.7124	7.650	6.008	1/2	12.250	9.6211	10.996	41.65	32.71
9/16	2.4414	1.9175	4.9087	8.301	6.520	9/16	12.691	9.9678	11.192	43.14	33.90
5/8	2.6406	2.0739	5.1051	8.978	7.051	5/8	13.141	10.321	11.388	44.68	35.09
11/16	2.8477	2.2365	5.3014	9.682	7.604	11/16	13.598	10.680	11.585	46.24	36.31
3/4	3.0625	2.4053	5.4978	10.41	8.178	3/4	14.063	11.045	11.781	47.82	37.56
13/16	3.2852	2.5802	5.6941	11.17	8.773	13/16	14.535	11.416	11.977	49.42	38.81
7/8	3.5156	2.7612	5.8905	11.95	9.388	7/8	15.016	11.793	12.174	51.05	40.10
15/16	3.7539	2.9483	6.0868	12.76	10.02	15/16	15.504	12.177	12.370	52.71	41.40

TABLE OF AREAS AND WEIGHTS OF SQUARE AND ROUND RODS AND CIRCUMFERENCES OF ROUND RODS

To avoid the necessity of referring to a separate handbook, the table on page 311 is given so that the area of cross-section of steel, obtained by multiplying the ratio of steel by the area of cross-section of beam above steel, may be readily transformed into mercantile sizes of round or square bars.

TABLES OF SAFE LOADS FOR BEAMS OF VARIOUS DIMENSIONS AND SPANS

The table on page 313 is calculated from item (15) of the table on page 302, so that the loading for various dimensions of ordinary beams of reinforced concrete, and, conversely, the dimensions of beams for any assumed loading, may be determined directly.

The table assumes a modulus of elasticity of concrete of 3 000 000, a working tensile strength of steel of 14 000 pounds per square inch, and a working strength of concrete in compression of 625 pounds per square inch.* Since the loads upon a beam are directly proportional to the values of K (see formulas (15) and (17), p. 299), the safe loads given in this table (p. 313) may be converted to any other assumption of modulus and strength. For example, if the concrete is known to have a modulus of elasticity of 4 000 000, and it is desired to limit the working strength of the steel to 12 000 and the working strength of the concrete to 625 pounds per square inch, inspection of items (9) and (15), page 302, indicates that the total safe load must be multiplied by a ratio of $\frac{80}{87} = 0.92$.

Examples 12 to 14, page 307, also illustrate the use of this table.

As high steel is not recommended for ordinary use on a small scale, no table is presented for safe loads for concrete reinforced with it. If the structure is of such size as to warrant its employment, the various members, as a matter of course, will require careful study, and the dimensions are readily obtained by calculation from the table on page 309.

BEAMS AND SLABS CONTINUOUS OVER SUPPORTS

In floors, reinforced beams and slabs are usually continuous over their supports or attached to walls. Although the practical effects of this have not been investigated scientifically, it is known that this decreases the bending moment at the center, and produces a negative bending moment at the ends. The supporting of slabs at four edges by means of cross girders or stiffeners further increases their strength.

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*625 pounds per square inch by the straight line theory adopted in this treatise corresponds to about 530 pounds per square inch by the theory of parabola distribution of compressive stress.

Safe Loading and Reinforcement for STONE CONCRETE BEAMS One Inch in Width. 1:2½:5 Concrete. Mild Steel.

From formula $w = \frac{(h-e)^2 K b}{1.5 l^2} = 58\left(\frac{d}{l}\right)^2$ Based on 0.69% steel and $K = 87$. (*See p. 312, and Item* (15) *p. 302.*)

Depth of Beam. (h) in.	Total Safe Load (w) per Linear Foot for Beam One Inch Wide including Weight of Beam. *For safe live load deduct weight of beam in column (22)* (See important foot-notes.)																			Weight of Beam one inch wide per linear foot. lb.	Depth to Steel. (d) in.	Depth below Steel. (e) in.	Steel Area in a Beam One Inch Wide. sq. in.	Safe Moment of Resistance. (*See p. 296.*) (M R) in.-lb.
	Span in Feet (l).																							
	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	25	30	35	(22)	(23)	(24)	(25)	(26)
5	37	26	19	14	12	9	8													5.4	4.0	1.0	0.028	1390
6	58	40	30	23	18	14	12	10												6.4	5.0	1.0	0.034	2180
7	84	58	43	33	26	21	17	14	12	11										7.5	6.0	1.0	0.041	3130
8	114	79	58	44	35	28	24	20	17	14	13									8.6	7.0	1.0	0.048	4260
9	139	97	71	54	43	35	29	24	21	18	16									9.6	7.75	1.25	0.054	5220
10	178	123	91	69	55	44	37	31	26	23	20	17								10.7	8.75	1.25	0.060	6660
11	221	153	113	86	68	55	46	38	33	28	24	22	19							11.8	9.75	1.25	0.067	8270
12	268	186	137	105	83	67	55	47	40	34	30	26	23	21						12.8	10.75	1.25	0.074	10060
13	307	213	157	120	95	77	63	53	45	39	34	30	27	24	21					13.9	11.50	1.5	0.079	11500
14	363	252	185	142	112	91	75	63	54	46	40	35	31	28	25	23				15.0	12.5	1.5	0.086	13590
15	423	294	216	165	131	106	87	73	63	54	47	41	37	33	29	26				16.0	13.5	1.5	0.093	15860
16	488	339	249	191	151	122	101	85	72	62	54	48	42	38	34	30				17.1	14.5	1.5	0.100	18290
17	557	387	284	218	172	139	115	97	83	71	62	54	48	43	39	35				18.1	15.5	1.5	0.107	20900
18	632	439	322	247	195	158	130	110	93	81	70	62	55	49	44	40				19.2	16.5	1.5	0.114	23680
19	670	466	342	262	207	168	139	116	99	86	74	66	58	52	46	42				20.3	17.0	2.0	0.117	25140
20		522	384	294	232	188	155	131	111	96	83	73	65	58	52	47	30			21.4	18.0	2.0	0.124	28190
22		644	474	363	286	232	192	161	137	118	103	91	80	72	64	58	37			23.6	20.0	2.0	0.138	34800
24			573	439	346	281	232	195	166	143	125	110	97	87	78	70	45			25.7	22.0	2.0	0.152	42100
26			682	522	412	334	276	232	198	170	148	130	116	103	93	83	53			27.8	24.0	2.0	0.166	50100
28				613	484	392	324	272	232	200	174	153	136	121	109	98	63	44		30.0	26.0	2.0	0.179	58800
30					561	455	376	316	269	232	202	178	157	140	126	114	73	50		32.1	28.0	2.0	0.193	68200
36						651	538	452	385	332	289	254	225	201	180	163	104	72	53	38.5	33.5	2.5	0.231	97640
42								628	535	462	402	354	313	279	251	226	145	101	74	44.9	39.5	2.5	0.273	135720
48									711	613	534	469	416	371	333	300	192	133	98	51.4	45.5	2.5	0.314	180100

RULES. 1. *For safe load of any width of beam* multiply by width ir

2. *For area of* (width of beam

3. *Total loads* lepth of steel a

4. *Total loads* ; ind same span

5. *For* 1 : 2 : 4 c 5 concrete add n (25), by
20% (see it oncrete has a h d increase
steel areas 302.)

6. *For* 1 : 3 : 6 c 7 concrete deduct 20% from safe loads and decrease steel areas, column (25),
by 20%.

End Reinforcement in Upper Part of Beam. The negative moment produced over the support by the loading of a continuous beam on both sides of this support produces tension in the upper portion of the beam. By the ordinary mechanical laws the maximum negative bending moment occurs when both portions of the beam are fully loaded.

Let

M'_B = bending moment at support for a beam loaded at center.
M''_B = bending moment at support for uniformly distributed load.
W = total load upon each beam in pounds.
w = uniformly distributed load in pounds per foot on each beam.
l = length of beam in feet.

The maximum negative bending moment in foot-pounds in a continuous beam at a support, due to central loading of the beams, may be considered as one-half the positive bending moment at the center of a single beam supported at its ends, or

$$M'_B = -\frac{Wl}{8} \text{ foot-pounds}$$

The maximum negative bending moment in foot-pounds at support, due to uniform loading of the beams, may be considered as two-thirds the positive moment at center of a uniformly loaded beam supported at the ends, or

$$M''_B = -\frac{wl^2}{12} \text{ foot-pounds}$$

To resist the tension due to this negative bending moment, it is advisable in continuous beams to place reinforcing metal just below the upper surface of the beam over the support and extending along the beams as far as the conditions require. The percentage of reinforcement is determined by the methods employed when the steel is in the bottom of the beam. The distance from the steel to the surface of the concrete is the same in the two cases.

Bending Moment at Middle of Continuous Beams. If the beams on both sides of the support are fully loaded, the bending moment for central loads (using above notation) is usually considered to be $\frac{Wl}{8}$ or one-half the moment of a beam supported at the ends, and the moment for uniformly distributed loading to be $\frac{wl^2}{24}$ or one-third the moment of a beam supported at the ends. However, the maximum bending in either of the beams will occur when it is loaded and the beam next to it is not loaded.

Mr. A Considère* states that French engineers assume that the safe minimum of the moment on the supports is only $\frac{1}{40}wl^2$, so that the resistance at the center of a beam with uniformly distributed load must be based upon a positive bending moment of $\frac{wl^2}{10}$, that is, the maximum bending moment in a continuous beam is taken as $\frac{4}{5}$ of the maximum bending moment in a beam supported at the ends. Engineers in the United States, also, have adopted this rule for calculations of strength of slabs. It is evidently on the side of safety, especially when the effect of the T-section, that is, the combination in one piece of the reinforced beam and floor slab, is considered.

However, since the effect of the T-section, which is analyzed in Appendix II, is somewhat problematical, and since the supports cannot be counted upon to be absolutely immovable, a moment of $\frac{wl^2}{10}$ is the lowest value which for the present should be adopted.

It is often required that the bending moment at middle of all uniformly loaded reinforced concrete beams shall be taken as though they were simply supported at the ends, that is at $\frac{1}{8}wl^2$, while continuous slabs are allowed the lower maximum moment of $\frac{1}{10}wl^2$. This plan has been followed in our beam table (p. 313) and slab tables (pp. 317 and 318).

If the moment is expressed in inch-pounds, the above values are multiplied by 12, and the bending moment for uniform loading on continuous slabs becomes

$$M''_B = \frac{6}{5}wl^2 \text{ inch-pounds} \qquad (19)$$

Haunches of concrete carried down to the lower flanges of steel I-beam girders produce similar effects as do continuity of the slabs, since the haunches lessen the deflection at the supports.

Flat Plates. Floor slabs are apt to be continuous both ways so as to form flat plates with supports on four sides. The increase in strength due to the support of the cross girder or stiffener is not usually allowed for in calculation, but if the slab is reinforced in both directions it considerably increases its strength and this increase is evident in tests of slabs so supported. The theory of flat plates is so intricate that calculations are not often attempted, but comparative tests will probably show that the strength of slabs or plates continuous in both directions is two or more times greater than the strength of slabs supported at their two ends, so that the dimensions and reinforcement may be reduced empirically. Mr. Joseph R.

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*International Engineering Congress, St. Louis, Missouri, 1904.

Worcester* in testing floor slabs of concrete reinforced with steel wire, found that the steel, if calculated by the usual theories, attained in one case an apparent tension of 250 000 pounds per square inch before rupture, thus showing (since this is an impossible stress) the evident inaccuracies of present theories to account for the stresses in continuous slabs.

Tests made upon plate glass at the Watertown Arsenal† show an average increase in strength of more than one-third by supporting a plate 16 inches square on its four edges instead of upon two edges. The ratio would have been still greater if the plates had been continuous over the supports.

The New York building laws, 1903, permit the calculation of the bending moment of square floor plates, reinforced in both directions and supported on four sides, by the formula, $M_B = \frac{Wl}{20}$ foot-pounds.

Cross Reinforcement in Slabs. Cross reinforcement, that is, steel rods parallel to the principal beams upon which the slab rests in addition to the principal bearing rods, is customarily used to prevent shrinkage and temperature cracks, and to give added strength. Although this reinforcement is not absolutely essential, if expansion joints are provided at frequent intervals, it greatly stiffens the floor and often renders the expansion joints unnecessary.

The metal required to provide for temperature changes may be calculated from the directions on page 378.

TABLES OF STRENGTH AND REINFORCEMENT OF SLABS

The table on page 317 is calculated with the assumption, stated on page 315, that the continuous length of the slab over the supports permits ½ greater loading than if the slab is simply supported at the ends. It is evident from the preceding paragraphs that this assumption is conservative, and it is probably safe, with careful designing, to allow double the given tabular loads on continuous slabs, supported on four sides and reinforced in both directions, thus conforming to the New York law.

The moment of resistance, M_R, which is in inch-pounds, is obtained either from formula (7) or formula (8) on page 298, whichever gave the lower moment of resistance. (See p. 310.) Formula (7) gave the lower moment for ratios of steel of 0.002 to 0.006, indicating that with these ratios the working strength of the slab depends upon the load upon the steel. Items corresponding to ratios 0.008 and 0.010 are calculated from formula (8), and consequently are limited by the working strength of the concrete in compression.

The most economical ratios of steel lie between 0.006 and 0.008.

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*See Journal Association Engineering Societies, 1905.

†Tests of Metals, U. S. A., 1901, p. 631.

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Safe Loading and Reinforcement of STONE CONCRETE SLABS One Foot in Width.

Proportions 1 : 2½ : 5.

From formula $w = \frac{5}{6}\frac{M_R}{l^2}$ (See p. 315.) Based on 0.2 to 1.0% Mild Steel. (*See p.* 316.)

Ratio of cross-section steel to beam above steel. (p*)	Total depth of slab. (h)	Total safe load (w) per square foot including weight of slab. For safe live load deduct weight of slab in Column (15). (See important foot-notes.) Span in feet. (l) 4	5	6	7	8	9	10	11	12	13	14	15	Weight of slab per square foot. lb. (15)	Depth to steel. (d) in. (16)	Depth below steel. (e) in. (17)	Steel area in a section of slab one foot wide. sq. in. (18)	See p. 298. Safe Moment of resistance (M_R) in.-lb. (19)
	2½	50	32											32	1¾	¾	0.042	970
	3	83	53											38	2¼	¾	0.054	1600
	3½	124	80	55										45	2¾	¾	0.066	2390
	4	174	112	77	57									51	3¼	¾	0.078	3340
0.002	4½	202	129	90	66									58	3½	1	0.084	3870
	5	263	168	117	86	66								64	4	1	0.096	5050
	6	411	263	183	134	103	81							77	5	1	0.120	7900
	7	592	379	263	193	148	117	95						90	6	1	0.144	11370
	8	806	516	358	263	201	159	129						103	7	1	0.168	15470
	2½	98	63	44	32									32	1¾	¾	0.084	1890
	3	163	104	72	53	40								38	2¼	¾	0.108	3120
	3½	243	156	108	79	61	48							45	2¾	¾	0.132	4670
	4	340	217	151	111	85	67	54						51	3¼	¾	0.156	6520
0.004	4½	394	252	175	129	98	78	63						58	3½	1	0.168	7560
	5	514	329	229	168	129	102	82	68					64	4	1	0.192	9870
	6	803	514	357	262	201	159	129	106	89				77	5	1	0.240	15420
	7	1157	741	514	378	289	229	185	153	129	110			90	6	1	0.288	22220
	8	1574	1008	700	514	394	311	252	208	175	149	128		103	7	1	0.336	30230
	2½	145	93	64	47	36								32	1¾	¾	0.126	2790
	3	240	154	107	78	60	47	38						38	2¼	¾	0.162	4610
	3½	358	229	159	117	90	71	57	47					45	2¾	¾	0.198	6880
	4	501	320	223	163	125	99	80	66	56				51	3¼	¾	0.234	9610
0.006	4½	581	372	258	190	145	115	93	77	64				58	3½	1	0.252	11150
	5	758	485	337	248	190	150	121	100	84	72			64	4	1	0.288	14560
	6	1185	758	527	387	296	234	190	157	132	112	97		77	5	1	0.360	22750
	7	1706	1092	758	557	427	337	273	226	190	162	130	121	90	6	1	0.432	32760
	8	2322	1486	1032	758	581	459	372	307	258	228	190	165	103	7	1	0.504	44590
	2½	175	112	78	57	44	34							32	1¾	¾	0.168	3360
	3	289	185	128	94	72	57	46	38					38	2¼	¾	0.216	5550
	3½	432	276	192	141	108	85	69	57	48				45	2¾	¾	0.264	8290
	4	603	386	268	197	151	119	96	80	67	57			51	3¼	¾	0.312	11570
0.008	4½	699	448	311	228	175	138	112	92	78	66			58	3½	1	0.336	13420
	5	913	584	406	298	228	180	146	121	101	86	74	65	64	4	1	0.384	17530
	6	1426	913	634	465	357	282	228	189	158	134	116	102	77	5	1	0.480	27390
	7	2054	1315	913	670	514	406	329	272	228	194	167	146	90	6	1	0.576	39440
	8	2796	1789	1243	913	699	552	447	370	311	265	228	199	103	7	1	0.672	53680
	2½	189	121	84	62	47	37							32	1¾	¾	0.210	3620
	3	312	199	139	102	78	62	50	41					38	2¼	¾	0.270	5080
	3½	466	298	207	152	116	92	74	62	52				45	2¾	¾	0.330	8940
	4	651	410	289	212	163	128	104	86	72	61			51	3¼	¾	0.390	12490
0.010	4½	755	483	335	246	189	149	121	100	84	71	61		58	3½	1	0.420	14480
	5	986	631	438	321	246	195	158	130	109	93	80	70	64	4	1	0.480	18920
	6	1540	985	684	502	385	304	246	204	171	146	125	110	77	5	1	0.600	29560
	7	2217	1419	985	723	554	438	355	293	246	210	181	158	90	6	1	0.720	42560
	8	3017	1931	1341	985	754	596	483	399	335	285	246	215	103	7	1	0.840	57940

*Percentages of steel are values in this column multiplied by 100.

RULES. 1. ***For load for any width of slab*** multiply by width in feet.

2. ***For area of cross-section of steel*** for any width of slab multiply column (18) by width in feet.

3. ***Total loads for other spans*** (*l*) and same depth of steel are inversely proportional to the squares of the spans.

4. ***Total loads for other depths of steel*** (*d*) and same span are proportional to the squares of the depths of steel.

5. ***For*** 1 : 2 : 4 ***Concrete*** or well-graded 1 : 2½ : 5 concrete add 20% to total safe loads and increase steel areas, column (18), by 20% (see Item (14), page 302). Or if the concrete has a high modulus and strength, add 12% to total safe load and increase steel areas by 12%. (See Item (4), page 302.)

6. ***For*** 1 : 3 : 6 ***Concrete*** or well-graded 1 : 3½ : 7 concrete deduct 20% from safe loads and decrease steel areas, column (18), by 20%. (See Item (16) or (26), page 302.)

A modulus of elasticity of concrete in compression of 3 000 000 pounds per square inch, a safe working compressive strength of concrete of 625 pounds per square inch, and a safe tension in the steel of 14 000 pounds per square inch, are assumed in the calculation of the table.

The values of *e* in column (17) are the distances from the center of gravity of steel to the bottom of the slab. In many cases it is advisable

Safe Loading end Reinforcement for CINDER CONCRETE SLABS One Foot in Width. Proportions 1 : 2½ : 5. *Mild Steel.* (*See p.* 319.)

From formula $w = \frac{5}{6}\frac{MR}{l^2}$ (*See p.* 315.) Based on 0.2 to 0.6% steel.

Ratio* cross-section steel to beam above steel.	Total depth of slab. (*h*)	Total safe load (w^1) per square foot including weight of slab. For safe live load deduct weight of slab in Column (12). (See important foot-notes.)							Weight of slab per square foot.	Depth to steel. (*d*)	Depth below steel. (*e*)	Steel area in a section of slab one foot wide.	Safe moment of resistance. (*See p.* 298.) (*MR*)
		Span in Feet (*l*).											
p	in.	4	5	6	7	8	9	10	lb.	in.	in.	sq. in.	in.-lb.
									(10)	(11)	(12)	(13)	(14)
0.002	2½	48	31						24	1¾	¾	0.042	920
	3	70	51	35	26				29	2¼	¾	0.054	1520
	3½	119	76	53	39				34	2¾	¾	0.066	2280
	4	166	106	74	54	41			39	3¼	¾	0.078	3180
	4½	192	123	85	63	48			43	3½	1	0.084	3690
	5	251	161	112	82	63	50		48	4	1	0.096	4820
	6	392	251	174	128	98	78	63	58	5	1	0.120	7530
	7	565	361	251	184	141	112	90	68	6	1	0.144	10840
	8	768	402	341	251	192	152	123	77	7	1	0.168	14750
0.004	2½	76	48	34	25				24	1¾	¾	0.084	1460
	3	125	80	56	41	31			29	2¼	¾	0.108	2400
	3½	187	120	83	61	47	37		34	2¾	¾	0.132	3590
	4	261	167	116	85	65	52	42	39	3¼	¾	0.156	5020
	4½	303	194	135	99	76	60	48	43	3½	1	0.168	5820
	5	396	253	176	129	99	78	63	48	4	1	0.192	7600
	6	619	306	275	202	155	122	99	58	5	1	0.240	11880
	7	891	570	306	291	223	176	143	68	6	1	0.288	17110
	8	1213	776	539	366	303	240	194	77	7	1	0.336	23290
0.006	2½	86	55	38	28				24	1¾	¾	0.126	1640
	3	141	90	63	46	35			29	2¼	¾	0.162	2710
	3½	211	135	94	69	53	42	34	34	2¾	¾	0.198	4050
	4	205	189	131	96	74	58	47	39	3¼	¾	0.234	5660
	4½	342	219	152	112	85	68	55	43	3½	1	0.252	6570
	5	447	286	199	146	112	88	72	48	4	1	0.288	8580
	6	608	447	310	228	175	138	112	58	5	1	0.360	13400
	7	1005	643	447	328	251	199	161	68	6	1	0.432	19300
	8	1368	876	608	447	342	270	219	77	7	1	0.504	26270

*Percentages of steel are values in this column multiplied by 100.

RULES.
1. ***For load for any width of slab*** multiply by width in feet.
2. ***For area of cross-section of steel*** for any width of slab multiply column (13) by width in feet.
3. ***Total Loads for other spans*** (*l*) and same depth of steel are inversely proportional to the squares of the spans.
4. ***Total loads for other depths of steel*** (*d*) and same span are proportional to the squares of the depths of steel.

to take these distances below the bottom of the steel instead of below the center of the steel (see p. 322), thus slightly increasing the thickness of the slab for the given loads.

The areas of section of patented rods vary from their nominal dimensions, and allowance must be made for this when determining the amount of steel to use.

Expanded metal, 3-inch mesh No. 10 gage, has a sectional area of steel of 0.185 square inches per foot of width; 6-inch mesh No. 4 gage has a sectional area of 0.259 square inches per foot of width. From these values, economical thicknesses of slabs and safe loading with this reinforcing material may be determined.

Slabs of Cinder Concrete. Cinder concrete floors may be designed by the table on page 318, which is based on a safe working compressive strength of cinder concrete of 225 pounds per square inch, a safe tension in the steel of 14 000 pounds, and a modulus of elasticity of cinder concrete of 850 000 pounds per square inch. It is noticeable that less steel can be used economically for a given thickness of slab than with broken stone or gravel concrete, because the strength of the slab is more apt to be limited by the strength of the cinder concrete than by the strength of the steel. The values for slabs with a ratio of steel of 0.002 are limited by the working strength of the steel, and the values with the higher ratios by the working strength of the cinder concrete.

TESTING FLOOR PANELS

The Prussian Regulations,* 1904, require:

> If a strip of a floor panel be cut out and tested by a trial load, this load shall be distributed uniformly over the whole strip and not exceed the weight of the floor and double the live load it is computed for. If such a strip is tested without being cut out of the panel, the test load shall be increased by one-half. Thus, if g denotes the dead load and p the live load, the test load will be for the former case $g + 2p$, and for the latter $1.5\,g+3p$.

STEEL IN COMPRESSION PORTION OF BEAM

In certain cases it is advantageous to place steel in the upper portion of a concrete beam or arch to provide for possible negative bending moment due to eccentric loading, and at the same time to assist in taking a share of the compression.

The steel, if imbedded, say, two inches, or properly hooped to prevent

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*[*Engineering Record*, July 2, 1904, p. 26.

buckling, may probably be counted upon to take a share of the compression in a ratio depending upon the relative moduli of elasticity of the steel and the concrete. Formulas for designing a beam or arch with steel in the compression side are presented in Appendix II.

PREVENTION OF DIAGONAL CRACKS IN BEAMS

The diagonal breaks which frequently occur in testing reinforced beams were formerly attributed to the failure of the concrete in shear, but recent tests indicate that the cracks, at least in part, are due to internal tension caused by a stretching and slipping of the rods employed in the reinforcement. If calculated as shear, beams have broken with an average end shear of 100 to 110 pounds per square inch.* As the ultimate strength of ordinary concrete in shear (see p. 270) appears to be at least 600 pounds per square inch, it is probable that the breaks are due chiefly to internal tension on the 45° lines.

To prevent these diagonal cracks, the steel should be so designed and placed as to give the greatest possible adhession to the concrete, and thus prevent slipping as the cross-section of the steel becomes reduced by its stretch. It is also customary as an extra precaution against shear and tension to place inclined or vertical rods at intervals in the beam either as separate stirrups or U-bars, or else, as in one or two patented systems, as a part of the longitudinal reinforcement. Still another plan consists of a vertical zigzag reinforcement, a single rod bent so as to form a series of V's with one leg of each loop inclined away from the center of the beam and the other leg vertical. The horizontal reinforcing rods rest in the bottom of the loops.

Theoretically, the slope of the reinforcement should be 45°, inclining away from the center of the beam, but because of the difficulty in placing rods at this angle, they are more frequently set vertically.

Location of Stirrups. The rule suggested by Mr. J. W. Schaub† for determining the metal to be used in the reinforcing stirrups in beams loaded uniformly is as follows:

Let

s = area of steel required in stirrup at any section of beam.

A = total sectional area of beam in square inches.

p = ratio of cross-section of horizontal steel to cross-section of beam.

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*Lewis J. Johnson in Journal Association Engineering Societies, June, 1904, p. 308.

†The formula with slightly different notation and the analysis from which it is derived is given in *Engineering News*, April 16, 1903, p. 348. From recent experiments, Mr. Schaub, in many cases where the horizontal reinforcement is prevented from slipping, questions the necessity for vertical reinforcement.

pA = area of section of horizontal steel in square inches.
l = length of beam in feet.
z = distance from end of beam to the section where stirrup is required.

Then

$$s = \frac{4\,pA}{l}\left(1 - \frac{2z + 1}{l}\right) \tag{20}$$

Mr. Schaub states: In a recent example, the metal in the horizontal plane was 0.03655 sq. in. per 1-inch width of beam. As the beam was 7 feet long, l was 7 feet. The metal required in the stirrups, one foot from the end, was $\frac{4 \times 0.03655}{7} \times \frac{4}{7} = 0.012$ sq. in. per 1-inch width of beam.

The spacing of the stirrups should be determined to a certain degree by the character of the loading. Some make it a rule to place them the same distance apart as the depth of the beam, or slightly closer than this, so that any diagonal line at an angle of 45° with the neutral axis will pass through the stirrups. As theoretical calculations of shear and internal tension give higher stresses near the ends of the beam, many designers place the bars there more frequently.

Mr. E. L. Ransome's empirical rule for spacing stirrups is to place the first a distance from the end of the beam corresponding to one-quarter the depth of the beam, the second a distance of one-half the depth of the beam beyond the first, the third a distance of three-quarters the depth of the beam beyond the second, and the fourth a distance of the depth of the beam beyond the third.

In some cases the size and location of the rods are calculated, as in a plate girder, as though the stress was actual shear, the Prussian requirements for 1904, for example, specifying that the shear shall not exceed 64 pounds per square inch.

DEPTH OF CONCRETE BELOW RODS

The selection of the thickness of the concrete below the rods is governed more by the proper fire and rust protection of the metal than by the stresses in the beam.

Prof. Charles L. Norton, who has made a careful study of the subject, considers a thickness of 2 inches essential for efficient fire protection. (See p. 433.) Since an excessive thickness adds to the danger of cracking, because the tension in the concrete increases with the depth below the steel, with but slight corresponding gain in strength to the beam, this

thickness, measured from the lower surface of the steel, and not from its center of gravity, may be taken as a maximum. Thus, in important members which are liable to severe fire, 2 inches may be considered the standard requirement, while for secondary members and floor slabs, a less thickness, ranging from ½ inch to 2 inches, is probably warranted.

The following thicknesses of concrete below the steel may be employed under ordinary conditions:

Thickness of Concrete below Steel.

Depth of slab or beam, inches.	Thickness below lower surface of rods,* inches.
1½ to 2	½
2½ to 4	¾
4½ to 8½	1
9 to 12	1¼
13 to 18	1½
19 to 20	1¾
Greater than 20	2

SPLITTING AT RODS

Tests of beams, and also failures of concrete beams in buildings due to unusual strains, have sometimes caused horizontal splitting on a plane with the rods. In some cases, at least, this is probably caused by the slipping of the steel, and is best prevented by securely anchoring the ends of the rods (this may be advisable even if the rods are patented bars of irregular cross-section), or by introducing stirrups or other vertical or inclined reinforcement. There is evidently greater danger of slipping where several bars are placed so close together that the thickness of concrete between them is small and the placing of the concrete between them difficult. An arbitrary rule may be suggested of spacing the rods no nearer together in the clear than the sum of their two diameters, and in no case less than 1½ inches apart, nor nearer than 1½ inches to either side of the beam.

Prof. McKibben has suggested a mathematical demonstration for determining the width of concrete required between the rods in order to make the resistance in shear equivalent to the adhesion of the concrete to the steel.

Let

L = length of rod considered.
P = distance in the clear between two rods.
D = diameter of rod.

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*Values up to depth of 20 inches are from tables of Mr. Edwin Thacher, except that his depths are taken below center of gravity of steel.

A = adhesion or bond between concrete and steel per square inch of surface of steel.

H = shearing strength of concrete per square inch.

If the beam splits at the rods, it is apt to shear through the concrete between the rods, and break the adhesion between the lower half of the rod and the concrete. When such splitting occurs, the shearing strength of the concrete between the rods, on a plane with their centers, is equal to or less than the adhesion of the concrete to the lower half circumference of one of the rods. Therefore, for a short length of rod, L, equate the strength in shear of the concrete between the rods to the adhesion between the concrete and the lower half circumference of the rod.

$$PLH = \frac{\pi DLA}{2}$$

$$P = 1.57\frac{A}{H}D \qquad (21)$$

If, for example, the ultimate adhesive strength, A, is assumed as 350 pounds per square inch, and the shearing strength is assumed as 400 pounds per square inch,* the formula becomes $P = 1.37\ D$, that is, the net distance between the surfaces of the rods is one and one-half times the diameter of the rod. This is a slightly smaller distance than is suggested in the empirical rule above.

ADHESION OF CONCRETE TO STEEL

The strength in adhesion of concrete and mortar to steel not only is of practical importance in reinforced concrete design, but also in the setting of bolts in mortar and concrete foundations. Tests by different experimenters upon the adhesion of smooth rods, based upon the surface area of contact of the steel and concrete or mortar, show extreme variations ranging from less than 100 pounds per square inch up to over 700 pounds. Where the yield point of the steel is not exceeded, the minimum ultimate adhesion for first-class concrete may be placed at about 275 pounds per square inch.† A factor of safety should be applied to this as to other stresses. Tests by Mr. R. Feret† and other experimenters indicate that the strength in adhesion is not only proportional to the area of the surface in contact, but depends upon the character of the surface and the nature of the concrete or mortar.

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*A low value for shear (see p. 270) should be assumed because the concrete is placed with greater difficulty between the rods, and may, therefore, be of lower strength than the rest of the beam.

†Christophe's Béton Armé, 1902, p. 476.

Experiments made by Mr. E. S. Wheeler* upon 1-inch diameter bolts, imbedded in mortar to depths varying from 1½ to 10 inches, gave an average result at the age of one month of 264 pounds per square inch of area of contact for 1:2 Portland cement mortar, and 111 pounds per square inch of area for 1:4 Portland cement mortar. The adhesive strength of mortar made from Portland cement and limestone screenings, Mr. Wheeler found in one series of tests to be double that of the mortar made with sand, averaging for 1:2 mortar about 510 pounds per square inch.

Mr. Feret† states that the adhesion of concrete to iron is nearly proportional to the percentage of cement in a unit volume of concrete. The best consistency for the concrete he considers to be so plastic as to be almost sloppy.

Prof. Hatt‡, with $\frac{7}{16}$-inch and ⅝-inch rods imbedded 6 and 6.4 inches respectively in 1:2:4 concrete obtained a strength in adhesion at about 34 days of 636 and 756 pounds per square inch of surface contact for the two diameters. He states that sliding friction after the adhesion was overcome was from 50% to 70% of the adhesion. Breaking the cubes with a hammer after the tests showed only partial contact between the rod and the concrete.

Experiments at the Massachusetts Institute of Technology under the direction of Prof. Charles M. Spofford,§ gave results upon the bond of union between steel rods and concrete shown in the table, page 325. The adhesion of 1:3:6 concrete to round and square rods varies, it is seen, from 219 to 274 pounds per square inch, depending upon the depth imbedded, while flat rods have lower adhesion than this. The patented rods, on the other hand, which are designated by the letters R, T, and J, and all of which have irregular surfaces, show a much higher bond of union.

Length of Rod to Imbed in Concrete. This greater adhesive strength of the patented rods appears to be due in part to the higher yield point and in part to the irregular surfaces. In the rods most deeply imbedded in proportion to their diameters, the stress reached a limit exceeding, by about 25% for plain steel and 40% for irregular surface steel, the yield point which might be expected in the steel, indicating that the slipping was due to the reduction in area of the rod. Comparing the specimens which failed below this highest stress in the rods, it is seen that the speci-

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*Report Chief of Engineers, U. S. A., 1895, p. 2941.

†Thonindustrie-Zeitung, 25 (153), 2, 213-2, 215, translated in *Cement*, July, 1902, p. 213.

‡Proceedings American Society for Testing Materials, 1902.

§*Béton & Eisen*, 3 Heft, 1903, p. 200.

mens with patented rods failed below the yield point of the steel when the area of surface of contact was less than about 50 square inches, corre sponding to a depth imbedded of about 25 diameters, and to an adhesive strength of about 425 pounds per square inch,* while the specimens with plain rods failed below the yield point of the steel when the surface of contact was less than about 80 square inches, corresponding to a depth imbedded of about 40 diameters and to an adhesive strength of about 270 pounds per square inch.* The more deeply imbedded rods apparently have less adhesive strength merely because failure is produced by the stretching of the steel and a consequent reduction in its diameter.

Tests of Bond of Union between Concrete and Steel. Proportions of Concrete 1:3:6. *Age, one month.* (*See p.* 324.)

By Charles M. Spofford.

Cross-section of Concrete. in.	Size of Rod. in.	Kind of Rod.	Depth Imbedded. in	Load at Failure. lb.	Adhesion. lb. per sq. in.	Pull in Rod. lb. per sq. in.	Depth Imbedded. in.	Load at Failure. lb.	Adhesion. lb. per sq. in.	Pull in Rod. lb. per sq. in.	Depth Imbedded. in.	Load at Failure. lb.	Adhesion. lb. per sq. in.	Pull in Rod. lb. per sq. in.
8 x 8	¾	Round	24	15300	271	34800	31	18600	255	42200	36	18600	219	42200
		Square	24	19700	274	35200	31	22600	243	40400	36	23900	221	42700
	1⅛ x ½	Flat	24	12400	159	22100	31	20300	201	36200	36	21700	185	38700
	1½ x ⅜	Flat	24	20300	226	36300	31	21700	188	38800	36	22130	164	39500
	2¼ x ¼	Flat	24	5000c	42	8930	31	25500	165	45500	36	26100	145	46600
6 x 6	½	R	12	12100a	454	48400	16	8100a	228	32400	26	16800	291	67200
	½	T	12	4850a	222	26900	16	8200	282	45500	26	10550b	223	58600
	½	J	12	12200a	508	87200	16	13120a	410	93700	26	13750b	264	98400
8 x 8	¾	R	20	25900	388	46300	24	31900	399	57000	36	36600a	305	63500
		T	20	21150	402	53000	24	18300a	290	45900	36	23700b	250	59400
		J	20	27600a	461	89100	24	25000a	347	80600	36	28000a	259	90500
10 x 10	1⅛	R	27	33100a	245	26100	37	26150	141	20600	50	34600a	138	27200
	1⅛	T	27	28100a	238	27300	37	48950a	304	47500	50	58450a	268	56700
	1⅛	J	27	40100	313	40100	37	44200	252	44200	50	57250	242	57250

Note: — R, T, and J are patented shapes. R and J rods have high elastic limit.
a Concrete split longitudinally before slipping.
b Rods broken.
c Injured specimen.

To provide for lack of homogeneity in the concrete and occasional poor workmanship in imbedding the rods, it is advised that the depth for imbedding rods with irregular surfaces shall be not less than 50 diameters and for plain rods not less than 80 diameters, and that the further precaution be taken, wherever possible, even with steel having an irregular surface,

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*These values are slightly below those resulting from tests in France. See Christophe's Béton Armé, p. 478.

of anchoring the ends or at places throughout its length, so that if the steel is at any time strained to its elastic limit, the beam will not suddenly fail by the rods pulling through.

Using the limit of 80 diameters, a ½-inch rod must be imbedded at least 40 inches (80 diameters) in each direction from the section of maximum bending moment.

In anchoring the ends of rods the washers or projections or bent portions should not only be of sufficiently large area to prevent crushing the concrete, but also should be stiff so as to avoid their bending and pulling through the beam or "backing out" at the end of the beam.

EXPERIMENTS UPON REINFORCED BEAMS

Tests upon reinforced concrete beams have been conducted at various universities in the United States, and by leading scientists in Europe. Valuable data with reference to the location of the neutral axis, the deformation and the ultimate loads with various percentages and classes of steel, have been recorded* in the United States by Professors Hatt, Howe, Lanza, Marburg, Talbot, and Turneaure, and in Europe by Messrs. Considère, von Emperger, Rabut, Ramisch, Ribéra, and Sanders.

Special results of many of these tests have been mentioned in the preceding pages.

Tests of Prof. Arthur N. Talbot. Tests of Prof. Talbot, although made with a leaner mixture of concrete than in the past has been customary (his proportions being 1:3:6, based on loose measurement of cement, or 1:3½:7, based on the unit of 100 pounds of cement per cubic foot), cover an exceedingly wide range of percentages of steel and types of steel. The beams were 15 feet 4 inches long, 12 inches wide, and 13½ inches deep, with the reinforcement 12 inches below the upper surface. These were tested on a span of 14 feet by two loads which divided the span into three equal parts. The exact proportions of the concrete were 96 pounds Portland cement to 3⅜ cubic feet sand to 6¾ cubic feet broken stone. The sand was well graded in size of grains and weighed 115 pounds per cubic foot loose and dry. The stone was Illinois limestone, with particles smaller than ¼ inch and coarser than 1½ inches screened out. The consistency was such that the water flushed to the surface under light ramming. The crushing strength of 6-inch cubes at the age of 60 days averaged 2 030 pounds per square inch. Various types of metal rods were used, most of them being placed horizontally, but a few being inclined or turned up at the ends.

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*See also References, Chapter XXIX.

Typical deformation and deflection curves are given in Fig. 89, page 288.

Prof. Talbot gives the following description of the manner of failure of each beam except those numbered 27, 22, and 28, which crushed at the top at maximum load:

> Vertical cracks through full width of beam every 4 to 8 inches of middle third. Load reached maximum and then dropped slowly. Lower fibers elongated rapidly, accompanied by the rapid widening of several cracks. After considerable further deflection concrete finally crushed out at top surface.

Tests of Reinforced Concrete Beams.

BY ARTHUR N. TALBOT. (*See p.* 326.)

Beam No.	Kind of Steel.	No. of Rods.	Size of Rods. in.	Area of Steel. sq. in.	Ratio of area of steel to beam above steel.	Maximum Load. lb.	Load Considered. lb.	Total Elongation of Steel. in.	Ratio of depth of steel to depth of neutral axis x			Estimated Total* Bending Moment. in. lb.	Moment of Resistance calculated from formula (7) or (8), p. 298. in. lb.	Remarks.
									As Measured.	Calculated by formula. (p.298)	Talbot's formula. (p.328)			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	
21	Round	3	½	0.59	0.0041	9 000	8 000	0.0665	0.34	0.33	0.33	261 000	226 890*b*	2 bars turned up
19	"	3	½	0.59	0.0041	9 200	9 200	0.0755	0.36	0.33	0.33	294 600	226 890*b*	2 bars turned up
16	Square	3	½	0.75	0.0052	9 900	9 900	0.065	0.37	0.36	0.35	313 200	284 700*b*	2 bars turned up
17	"	3	½	0.75	0.0052	10 000	9 500	0.059	0.37	0.36	0.35	302 000	284 700*b*	2 bars turned up
27	"	4	¾	2.25	0.0156	26 900	25 000	0.066	0.53	0.54	0.54	725 500	774 000*a*	2 bars turned up
9	Ransome	3	½	0.75	0.0052	22 800	18 000	0.142	0.34	0.36	0.35	540 000	474 500*c*	8 stirrups
15	Thacher	3	¾	1.20	0.0083	18 400	15 500	0.0715	0.41	0.43	0.41	466 000	443 300*b*	2 bars turned up
10	"	3	¾	1.20	0.0083	16 600	14 500	0.065	0.43	0.43	0.41	438 000	443 300*b*	2 bars turned up
22	Kahn	3	¾	2.40	0.0167	24 400	22 000	0.064	0.57	0.55	0.56	641 000	786 200*a*	Bars sheared up
4	"	5	½	2.00	0.0139	23 000	21 000	0.069	0.47	0.52	0.51	615 000	714 800*b*	Bars sheared up
14	"	4	½	1.60	0.0111	17 200	17 000	0.062	0.46	0.48	0.46	505 500	580 400*b*	Bars sheared up
5	"	3	½	1.20	0.0083	15 000	13 000	0.0625	0.42	0.43	0.41	396 000	443 200*b*	Bars sheared up
28	Johnson	6	¾	2.19	0.0152	34 300	31 000	0.101	0.53	0.53	0.53	893 500	768 700*a*	4 bars turned up
13	"	7	½	1.40	0.0097	29 000	27 500	0.111	0.45	0.46	0.43	800 500	681 400*a*	4 bars turned up
20	"	5	½	1.00	0.0069	20 900	20 000	0.132	0 44	0.41	0.39	593 500	615 600*a*	3 bars turned up
2	"	5	½	1.00	0.0069	20 600	19 000	0.119	0.39	0.41	0.39	565 500	615 600*a*	Horizontal bars
7	"	3	½	0.60	0.0042	14 000	13 000	0.1175	0.33	0.33	0.33	401 000	384 400*c*	Horizontal bars
3	"	3	½	0.60	0.0042	14 000	12 000	c 1065	0.31	0.33	0.33	373 000	384 400*c*	2 bars turned up
								Average	0.418	0 422	0.411	506 906	507 388	

NOTE: — Columns (6) (11) (12) and (14) have been added by the authors.

*As calculated by Prof. Talbot. Based on "Load Considered" column (8).

a. Based on crushing strength of concrete of 2 030 lb. per square inch because the moment thus obtained is lower than the moment based on yield point of steel.

b. Based on yield point of steel as 36 000 lb. per square inch.

c. Based on yield point of steel as 60 000 lb. per square inch.

A portion of the data resulting from the experiments is tabulated above. Column (10) is taken from a separate table of Prof. Talbot's.*

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*University of Illinois Bulletin, September, 1904.

and columns (11) (12) and (14) are added by the authors to compare the actual tests and the theory adopted in this treatise.

Prof. Talbot suggests an empirical straight line formula* for the location of the neutral axis with different percentages of steel, which avoids the more intricate calculations necessary with the usual theoretical formulas involving the modulus of elasticity. Adopting the same notation employed throughout this treatise (see p. 295), let

x = ratio of depth of neutral axis to depth of center of gravity of steel.

p = ratio of area of section of steel to area of section of beam above center of gravity of steel.

Then

$$x = 0.26 + 18\,p \tag{22}$$

Column (12) gives values of x calculated from this formula. It is probable that the formula may be adapted to concrete of other strength and elasticity by changing the values of the constants.

One of the most important conclusions which, in the authors' opinion, may be drawn from Prof. Talbot's tests, is the fact that computations made by the ordinary theory adopted in this treatise produce values for the neutral axis, and also for the ultimate moment of resistance, which are so near to the experimental results that these theoretical formulas (see p. 293) may be employed with confidence.

Calculating the location of the neutral axis by formula (6), page 298, and employing a ratio of the moduli of elasticity of steel to concrete of 20, — which Prof. Talbot's tests† of elasticity show to be an average value between loads of 1 000 and 1 700 pounds per square inch, stresses which correspond to the compression in the beam when the neutral axis is as given, — the theoretical distances given in column (11) agree almost exactly with the actual measurements in column (10). The moments of resistance calculated in column (14) also agree closely with the total bending moments in column (13).

REINFORCED COLUMNS

The practical advantage of the introduction of vertical steel rods into columns for the purpose of taking a portion of the compression has been questioned by some designers, but recent tests‡ on long columns indicate that the imbedded steel may be counted upon to take its portion of the

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*Prof. Talbot gives the derivation of this formula and a theoretical discussion of his tests in Journal Western Society of Engineers, August, 1904.

†Journal Western Society of Engineers, August, 1904.

‡Gaetano Lanza in Transactions American Society Civil Engineers, Vol. L, p. 483.

loading. The elastic limit of the steel may be reached without danger of buckling if the steel is placed at least 2 inches from the surface.

Occasional hoops spaced at distances apart not less than the width of the column are an added precaution against buckling of the rods and probably add stiffness to the column. The size and location of such hoops are discussed in connection with column design on page 465.

In combinations of steel and concrete designed to resist compression it is customary to assume that the load is borne by the two materials in a ratio determined by their relative moduli of elasticity.

Let

C_1 = total unit compression upon concrete and steel (*i.e.*, the total load divided by the combined area of concrete and steel) in pounds per square inch.

C = unit compression in concrete in pounds per square inch.

p = ratio of cross-section of steel to total cross-section of column.

$r = \frac{E_s}{E_c}$ = ratio of moduli of elasticity of steel to concrete.

Then

$$C_1 = C\,[(1 - p) + rp] \qquad (23)$$

$$p = \frac{C_1 - C}{C(r - 1)} \qquad (24)$$

The following example illustrates the use of these formulas.

Example 1. What percentage of reinforcement should be introduced into a column which must be designed to carry a load of 650 pounds per square inch when the working stress upon the concrete is limited to 500 pounds per square inch?

Solution. Assuming a ratio of elasticity of concrete to steel of 10, and substituting the values in formula (24), gives

$$p = \frac{650 - 500}{500(10 - 1)} = 0.033$$

Hence, 3.3% of steel should be introduced into the column to assist in bearing the compression. A 20-inch column would thus require 400 × 0.033 = 13.2 square inches of steel, which corresponds to a single round rod $4\frac{1}{8}$ inches in diameter, or 4 rods each $2\frac{1}{16}$ inches in diameter.

Example 2. What load will a concrete column 24 inches square, reinforced with four $2\frac{1}{2}$-inch vertical round rods, safely carry if the compression in the concrete is limited to 450 pounds per square inch?

Solution. The area of four $2\frac{1}{2}$-inch rods from the table on page 311 is

$4.9 \times 14 = 19.6$, which corresponds to a ratio of steel, p, of $\frac{19.6}{24 \times 24} = 0.034$. Substituting the values of C and p in formula (23) and assuming a ratio of elasticity, r, of 10, $C_1 = 450\ [(1-0.034) + 10\ (0.034)] = 588$ lb. per sq. in. The total allowable load is therefore $588 \times 576 = 339\ 000$ pounds, or 169½ tons of 2 000 pounds.

SYSTEMS OF REINFORCEMENT

One of the earliest recorded examples of the application of reinforced concrete is a boat of concrete and iron, built by Mr. L. J. Lambot in France, and shown at the Paris International Exhibition in 1855.[*] In 1861 Mr. Coignet began his investigations, and in 1866 Mr. Monier, to whom the invention of reinforced concrete is often attributed, applied the combination of concrete and iron to various structures, and laid the foundation for its future widespread applications.

As long ago as 1877, Mr. W. E. Ward,[†] at Port Chester, N. Y., built a house entirely of concrete, reinforced with iron I-beams and round rods.

The rapid development of reinforced concrete has resulted in the introduction of numerous systems, many of them covered by patents, for arranging the metal in the concrete, or for special forms of metal. These systems are fully described in the various French works on reinforced concrete.[‡]

A few of the systems, representing both the arrangement and the form of the metal, are described below, and forms of metal extensively used in the United States are illustrated in Fig. 91.

Systems of Reinforcement

Bonna. Metal of cruciform cross-section.

Chaudy and Degon. Cross rods passing under bearing rods, but looped up between them.

Coignet. Round bars in top and bottom of beam connected by diagonal wire lacing.

Columbian. Vertical steel plates with horizontal ribs.

Cottacin. Round rods interlaced in the same manner as in wire netting.

Cummings. Bars of different lengths having their ends bent to an incline and formed into a loop to resist internal stresses.

De Man. Undulated Bars. (See Fig. 91.)

Donath. Inverted T-beams or I-beams connected by horizontal diagonals of light, flat metal on edge.

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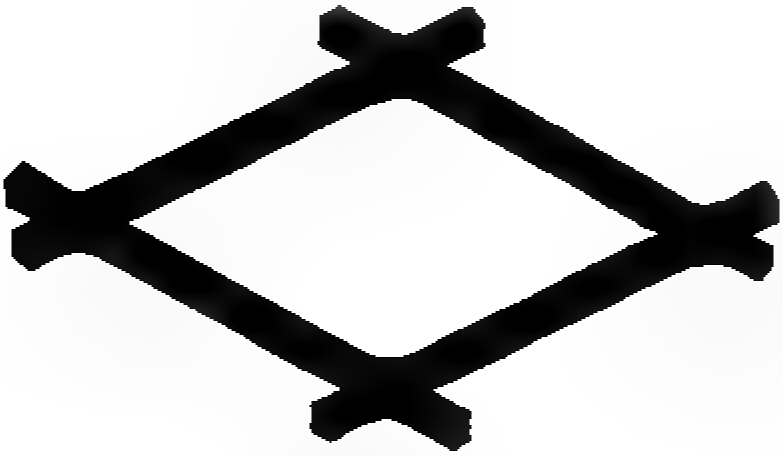
[*] Christophe's Béton Armé, 1902, p. 1.

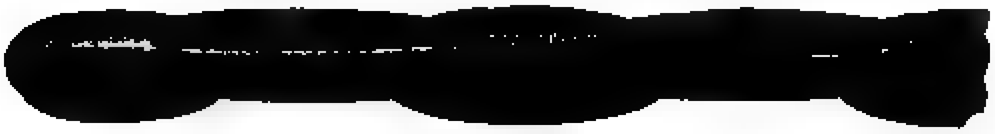
[†] Transactions American Society of Mechanical Engineers, Vol. IV, p. 388.

[‡] See among others Christophe's Béton Armé, 1902, pp. 10–71, and Morel's Ciment Armé, 1902, pp. 88 to 152.

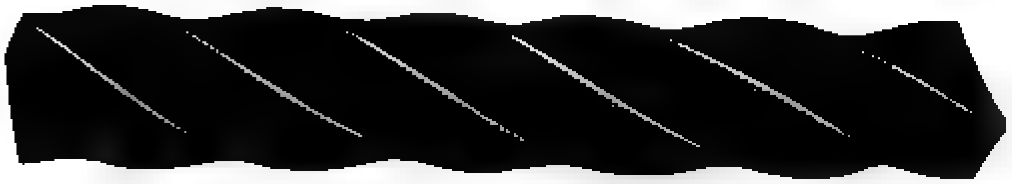


Expanded Metal.

Kahn Trussed Bar.



Thacher Bulb Bar.



Ransome Twisted Bar.

Johnson Corrugated Bar.

De Man Undulated Bar.

Fig. 91.—Types of Reinforcing Steel. (See pp. 330 and 332.)

Expanded Metal. Sheet steel, slit and expanded, so as to form a diamond mesh. (See Fig. 91, p. 331.)

Habrich and Düsing. Flat metal twisted hot.

Hennebique. A combination of alternate straight bars and bars with ends bent up at an angle, with vertical U-bars, or stirrups, of flat iron passing around the straight bars and reaching nearly to the top of the beam.

Holzer. Metal in form of I-beams.

Hyatt. Flat plates or bars set on edge and pierced with holes through which pass small round rods to form the cross reinforcements.

Johnson. Corrugated bars. (See Fig. 91, p. 331.)

Kahn. Horizontal flanged bars with flanges sheared up at intervals. (See Fig. 91, p. 331.)

Lock-Woven Steel Fabric. Steel wire mesh, locked at intersections.

Melan. Steel ribs, either I-beams or 4 angles latticed, imbedded in the concrete of the arch.

Monier. Two series of round parallel bars at right angles to each other.

Parmley. Bars with bent ends, to place in the sides of a conduit or the haunches of an arch to resist tension.

Rabitz. Various combinations employing galvanized wire.

Ransome. Square steel rods twisted cold. (See Fig. 91, p. 331.)

Roebling. Flat steel bars set on edge, clamped to supporting beams, and held in alignment by flat bar separators.

Schüllter. Like Monier System except rods are placed diagonally.

Thacher. Bulb bars. (See Fig. 91, p. 331.)

Visintini. Beams of concrete, cored out so as to form lattice girders.

CHAPTER XV

PREPARATION OF MATERIALS FOR CONCRETE

The various operations relating directly to the laying of concrete are discussed in detail in this and several succeeding chapters. While the selection of the special methods and machinery, which are described at length in the succeeding chapters, are determined by local conditions, certain general principles apply to all classes of work. The preparation of the materials relates to the storing of cement, the screening of sand and gravel, and the crushing of stone.

STORING CEMENT

Portland cement is not injured by storing in a dry place for an indefinite length of time; in fact, contrary to former belief, instead of deteriorating, the quality is often improved by storage. Cement manufacturers when rushed with orders sometimes ship material which, not being sufficiently air-slaked, contains free lime that exposure to air may change to a hydrate and thus render harmless.

Recognition of the fact that exposure to dry atmosphere does not injure cement has led to packing it in bags instead of in barrels, thus saving both the cost of the barrel and the extra freight upon it. If, however, the work is in a damp location, as in marine construction, barrel shipments are advisable.

The economy of storing the cement as near as possible to the mixing platform or mixing machine is obvious, but since, on the other hand, it is more easily handled and is always less in volume than sand and stone, these should be given the preference in the matter of location.

SCREENING SAND AND GRAVEL

The three most common methods of screening are (1) by hand, that is, by throwing shovelfuls of the material on to an inclined screen, (2) by dumping or hoisting the material on to a fixed inclined screen, (3) by a revolving screen.

Cost of Hand Screening. The cost of hand screening depends upon the total amount of material handled rather than upon the quantity of sand or gravel produced. A material most of whose particles run through the screen can be most cheaply screened, because the screen can be moved,

or arranged over a hole, while if a large proportion of the particles are caught they must be shoveled from the foot of the screen.

An average laborer, properly superintended, will throw about 24 cu. yd. of material against a screen in a ten-hour day, but in estimating the cost, allowance must be made for shoveling the material out of the way, moving screen, and superintendence.

The following are approximate costs of screening sand and gravel by hand under ordinary conditions. The prices are from actual records on a number of jobs and are based on labor at $1.50 for ten hours, with a suitable allowance for superintendence and contractor's profit. The minimum prices apply to first-class men.

	Average cost per cu. yd.	Minimum cost per cu. yd.
Screening sand, coarse stuff wasted	$0.11	$0.08
Screening gravel to remove large stones	0.15	0.10
Screening gravel to remove sand, sand wasted	0.24	0.17
Screening gravel coarse, and fine stuff, both measured	0.18	0.12

If laborers are working alone with no foreman in sight, as is often the case on concrete work, 50% should be added to the average costs.

Inclined Screen fed by Carts, Derrick Buckets, or Endless Chain. The slope of an elevated screen may vary from 35° to 45° from the horizontal, according to the character of the material. Coarser screens are required to pass material of a certain size than for hand screening.

At the new Cambridge Bridge, Boston, the contractors employed a screen about 15 feet long, hinged at the top so that the slope could be varied to suit the material. A hopper located above the screen fed on to a 3-inch bar screen, consisting of parallel iron bars about 3 inches apart, supported by iron cross pieces about 5 inches apart. The stones too large for the concrete ran down this coarse screen, and rolled off one side, while the remainder of the material fell through it on to a screen with 1-inch by ¾-inch mesh, which separated the medium gravel from the sand.

On another large job in Everett, Mass., where an inclined screen was fed by a bucket elevator supplied by carts, 300 to 350 cu. yd. of sand and gravel were screened in ten hours, and an even larger quantity could have been handled had it been supplied with absolute regularity.

The cost of screening by this method depends both upon local conditions and the quantity screened. The average cost may be assumed to be from 4 to 8 cents per cubic yard when large quantities of sand or gravel are handled at once.

Rotating Screens. Rotating screens, cylindrical or hexagonal in shape, although most frequently employed for separating crushed stone

(see p. 339), are also adapted, if power is available, for separating sand from gravel, or for separating gravel into several sizes to remix in the theoretical proportions required for a dense, impervious concrete.

While the first cost of a rotating screen is more than that of an inclined screen, less elevation is required and it may be fed with a bucket conveyor.

A plant for ordinary concrete made from two aggregates, sand and gravel, requires a screen with only two sizes of mesh, the smaller about ⅜-inch and the larger 2, 2½ or 3-inch mesh, as desired. Often no screening is required except to remove the sand, as a few large stones do no harm. The screen may be about 3 feet in diameter by 12 feet in length.

The present tendency, for concrete which is to be subjected to severe stress or to water pressure, is to require more scientific proportioning by separating the aggregate into several sizes and remixing them so as to produce the greatest density. This separation may be accomplished in practice by adding more sections, and thus lengthening the screen, or by employing a double cylinder, which occupies about half the space of a single cylinder.

The inner cylinder of a double-cylinder screen is composed of two or more sections of different sized mesh, and the outer cylinder is composed of two or more corresponding sections which are entirely separate from each other so that each may discharge into a separate bin. Each outer section has a finer mesh than the corresponding section of the inner cylinder. The material, after passing through a section of the inner cylinder, falls upon the outer wire and is again separated, the part which is caught rolling out through an annular opening into one bin and the remainder passing through the mesh into another bin.

STONE CRUSHING

The crushing of stone for concrete must be approached from a different standpoint than the preparation of material for macadam paving, although the costs will not vary materially from those of a well-arranged portable crushing plant used on road construction.

For city or town macadam paving, where a suitable ledge is available, it is possible to establish a fixed plant with stationary engine, large stone bins, and economical machinery for handling cars, so that the stone can be hauled over a system of movable tracks directly from the ledge to the crusher, while for country road building the plant is arranged with a view to its portability, sometimes even resting on wheels.

For concrete work a plant intermediate in style between these is usually required. Its design is governed by the local conditions and by the quan-

tity of concrete to be made. In some cases where the concrete is laid in excavation it is possible to locate the crusher on the bank, and allow the stone to pass by gravity on to and through an inclined screen, or, if "crusher run" is used, to fall directly into a pile below. Generally the stone from the crusher must be taken by bucket or belt conveyors to bins, located, if possible, above the concrete mixer, or where the stone can be conveniently conveyed to the mixer without shoveling.

Name and Number of Parts.

1 Main Frame
2 Round Shaft
3 Fixed Jaw Plate
4 Swing " "
5 Swing Jaw
6 Pitman
7 Toggle Block
8 Wedge
9 Eccentric Shaft
10 Swing Jaw "
11 Upper Half Cheek Plate
12 Lower " " "
13 Bolt for " "
14 Toggle
15 Toggle Bearing
16 Bolt for Wedge
17 Bolt for Toggle Block
18 Cover – Main Bearing
19 " " Swing Jaw Shaft
20 Grease Cup
21 Balance Wheel
22 Bolt for Swing Jaw Shaft cover
23 " " Main Bearing
24 Pulley
25 Grease Box Cover
26 Bolt and Thumb Screw
27 Bolt for Swing Jaw Plate
28 Shackle Pin
29 Spring Rod Shackle
30 Spring Rod
31 Spring Bar
32 Washer
33 Washer
34 Hand Wheel
35 Thumb Nut
36 Rubber Spring
37 Bolt for Pulley
38 Grease box cover
for Main Bearing

FIG. 92.—Jaw Crusher. (See p. 336.)

Stone Crushers. Stone crushers are of two general types, jaw crushers and gyratory crushers.

The size of a jaw crusher is designated by the opening into which the stone is introduced. A 16 by 10-inch crusher has jaws 16 inches in width, and the space between the two jaws at the top is 10 inches. A "duplex" crusher has two pairs of jaws operated by the same shaft, but working alternately by means of different eccentrics. Single jaw crushers range in size from 3 by 1½ inches to 36 by 24 inches.

The operation of a typical jaw crusher is shown in Fig. 92. One of the jaws is fixed, and the other is hinged at the top, and swung back and forth

through a very small arc. The motion is imparted by the eccentric shaft, which, in revolving, raises and lowers the "pitman," whose lower end is connected by toggles with the lower end of the movable jaw. The size of the stone passing through the jaws, that is, the size of the largest particles, is regulated by the opening at the bottom of the swing jaw, which is changed by using longer or shorter toggles.

The capacity of any crusher — that is, the quantity of broken stone which it will turn out per hour or per day — is dependent not only upon the size of the crusher, but upon the texture of the stone and the sizes of the largest particles. From the following catalogue capacities for a 16 by 10-inch jaw crusher per day of ten hours, it may be inferred that the quantity turned out is nearly in the ratio of the sizes of the stones.

120 tons crushed to 2½-inch size
100 " " " 2 " "
80 " " " 1½ " "
60 " " " 1 " "

In estimating the actual daily output of a crusher, — and this is in fact true for most machinery, — all catalogue figures are likely to be misleading because they are based on maximum capacity with continuous feeding, while in practice there are likely to be unavoidable delays. An average day's work of ten hours, — based on actual records obtained by the authors from a number of jobs, — for a 15 by 9-inch crusher set for 2½-inch stone, with a small percentage of tailings, may be taken at 65 cu. yd. or, say, 78 tons, in ten hours. This estimate applies to continuous running of the crusher, allowing only for occasional unavoidable delays.*

A section of a gyratory crusher, which is adapted for more stationary plants, is shown in Fig. 93, page 338. It consists essentially of a cone with a gyratory motion within an inverted conical chamber or shell. The size of the crusher is determined by the width of the opening between the top of the cone and the shell, and the circumference. The gyratory motion of the cone shaft is produced by an eccentric keyed to its lower end. As the shaft revolves, the cone is given a kind of a rocking motion which continually directs it toward, and then away from, different portions of the shell. The size of the broken stone is regulated by raising or lowering the cone on the shaft.

For a concrete plant producing 200 cubic yards per day, manufacturers recommend a No. 4 gyratory crusher with openings 8 x 27 inches.

The horse-power required to drive a crusher and its attendant machinery

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*The Annual Report of the Newton, Mass., City Engineer for 1891 gives interesting data on detail costs of stone crushing, a portion of which are here summarized on page 343.

varies largely with the material handled. It is advisable to make ample allowance above the figures given in manufacturers' catalogues. It is, also, economical to use a wider and heavier belt than is generally specified,

Fig. 93.—Gyratory Crusher. (See p. 337.)

in order to avoid delays and shutdowns. When ordering almost any kind of machinery the authors make it a practice to require a wider and heavier pulley than the standard width. It is wise to make a pulley at least 2 inches wider than the belt which is to be run upon it.

Crusher Screens and Bins. A typical design, by Mr. Earle C. Bacon, for bins suitable for a plant where the concrete mixer or mixing platform is located at a distance from the crusher is shown in Fig. 94. With slight changes they may be arranged to discharge into hoppers over a concrete mixer. The dimensions of timber employed in the construction may be used as a basis for bins of other sizes.

Fig. 94.—Small Crushing Plant with Elevator, Screen, and Portable Bins. (See p. 339.)

A safe slope for the bottom of stone bins is 45°, although if lined with sheet iron this may be decreased to 35° or 40°.

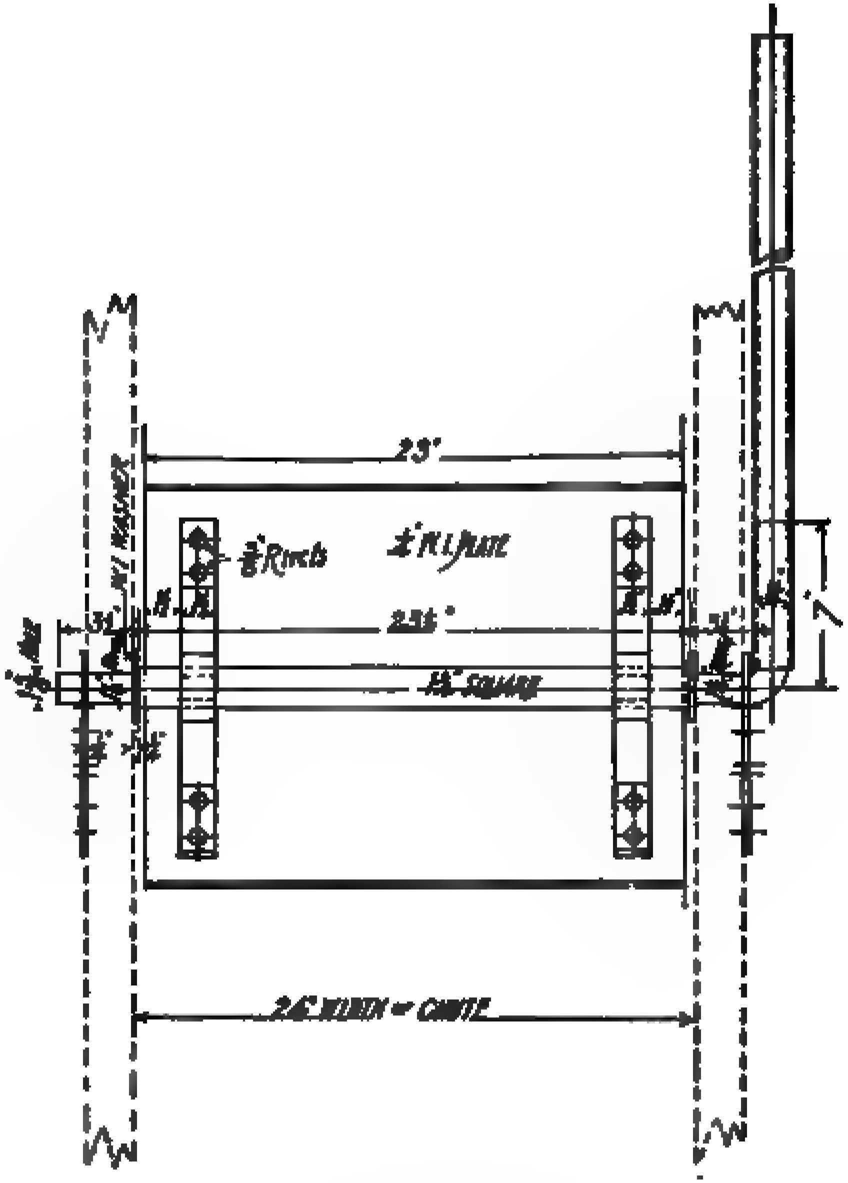
Screens for broken stone as shown in Fig. 95, page 340, are usually made in sections varying in length from 3 to 5 feet, so that they can be bolted together and give as many divisions of sizes as are required. The diameters vary from 24 to 48 inches. The mesh of a rotating screen should be about 20% smaller in diameter than the required size for the stone, as there is more or less wear on the screen, which enlarges the holes, and this allowance will also assist in excluding the oblong pieces whose longest dimen-

sion is above the limit. For concrete, unless two or more sizes of stone are mixed, no more than two sizes of mesh are required, one, ¼-inch to remove the dust, and the other, 2, 2½, or 3-inch to remove the coarse stuff. Often it is necessary only to remove the dust which may then be used as sand.

Stone Bin Gates. A gate designed by Mr. C. S. MacHenry, of the Greene Consolidated Copper Co., has proved extremely satisfactory for cutting off the flow of materials of the nature of broken stone, gravel, and sand. A detail drawing of this is shown in Fig. 96.

Cost of Stone Crushing. The cost of stone crushing is so dependent upon local conditions and upon the character of the rock, that only approximate estimates based upon actual experience can be given. There are, in general, two classes of work, — one where the rock is blasted from a ledge near at hand, and the other where the crushers are supplied with boulders or other loose rock. The gang at the crusher is similar in both cases, and the chief difference in operation is the extra gang for drilling and breaking up the stone in the ledge. On the other hand, usually more permanent, and therefore more economical, arrangements for hauling the stone can be made in ledge excavation than when the stone is obtained from various sources.

FIG. 95 — Rotating Screen. (*See p.* 339.)



18"X24" SWINGING GATE —
FOR STONE OR SAND BINS.

. 96.—Gate for Stone or Sand Bins. (*See p.* 340.)

A typical gang* for operating a 15 by 9-inch crusher, turning out, say, 65 cubic yards of broken stone in ten hours, is as follows:

One foreman.
One engineman.
Two men feeding crusher.
One other man at crusher on odd work.
Three men loading stone into carts to supply crusher.
Two single carts with one teamster hauling stone to crusher.

The number of teams required to haul stone to crusher depends, of course, upon the length of haul. Sometimes additional men will be needed to pass the stone to the men feeding the crusher; on the other hand, if the stone is dumped directly into a hopper above the crusher so as not to require handling, two men are capable of supplying a crusher whose capacity is 200 cubic yards per day.

The labor of drilling a ledge obviously depends upon the quality and seaminess of the rock and the depth of the holes. Under ordinary conditions, a steam drill with two men can be counted upon to loosen considerably more rock than can be handled by a 15 by 9-inch crusher. The cost of barring out and sledging the blasted rock may be estimated on the basis of about 10 cubic yards (measured after crushing) per man per day of ten hours. If the crusher is a large one, say a No. 6 rotary (11 by 36 in.), a man will bar and sledge about double this quantity because it does not need to be broken so fine. The figures are averaged by the authors from actual observed speeds on a number of jobs.

In estimating the cost of crushing stone, the original cost of the plant is an important item. The allowance for this per yard of rock is dependent upon the length of time the plant is to be operated, and the probable value of the machinery when the work is complete, as well as upon the interest on the investment and the cost of repairs. A plant similar to that shown in Fig. 94, page 339, with a 16 by 10-inch jaw crusher, may be estimated to cost from $2,000 to $2,500.†

A very careful analysis of the actual cost of crushing stone for macadam in a large gyratory crusher was made by Mr. Albert F. Noyes, City Engineer of Newton, Mass. His prices are based on common labor at $1.75 per day of nine hours, drill men at $3.00, drill helpers at $1.75, engineman for crusher at $2.00, and two one-horse carts with driver at $5.00. The detail costs per cubic yard of crushed stone were as follows:

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*Actual gang employed on a concrete contract for the Metropolitan Water Works, Mass.

†Estimated by Earle C. Bacon.

Cost per cubic yard of Quarrying and Crushing Hard Green Trap at Newton, Mass.[*]

Labor of steam drilling	$0.092
Coal, oil, waste, powder, drilling and repairs for drilling and blasting	0.084
Sharpening drills and tools	0.069
Breaking stone for crusher	0.279
Filling carts with rough stone	0.098
Carting stone to crusher	0.072
Feeding crusher	0.053
Engineman of crusher	0.031
Coal, oil, and waste for crusher	0.079
Repairs	0.041
Total cost per cubic yard of crushed stone	$0.898

The total cost of crushing in a jaw crusher conglomerate ledge stone drilled by hand, Mr. Noyes gives as $1.113 per cubic yard; of trap cobble stone wheeled to crusher in barrows, as $0.445 per cubic yard; and of granite cobble stone hauled in carts, as $0.372 per cubic yard.

These costs, which, as well as the wages paid per day, must be taken into account when estimating under other conditions, are based upon an output per hour of 7.7 cubic yards hard green trap, 8.9 cubic yards conglomerate ledge, 11.8 cubic yards trap cobble stone, and 9 cubic yards granite cobble stone.[†]

Data on Broken Stone. Broken stone is often sold by weight instead of by the cubic yard, because of the variation in volume due to handling or transporting. A cubic yard of broken trap stone may vary in weight from 2 400 to 2 700 pounds.[‡] If measured after carting some distance, broken stone will weigh about 10% heavier per cubic yard than at the crusher, because of the settling. The authors have found by repeated measurements that 100 pounds per cubic foot is a fair average weight for screened trap rock after it has been shaken down by hauling, although when measured loose in a small measure an average weight is about 90 pounds. Crusher run stone is about 10% heavier than this because it contains less voids. Stones having lower specific gravities than trap are correspondingly lighter in weight.[§]

On macadamized or paved roads, if no steep hills are to be encountered, two horses will haul from 6 000 to 7 000 pounds of broken stone to a load. Very high side boards are of course necessary to carry this quantity.

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[*] Annual Report of City Engineer for 1891.

[†] Cost per cubic yard of stone crushing for pavement in various towns is given in Report Mass. Highway Commission, 1895, p. 38, and further data in *Engineering News*, March 27, 1902, p. 258, and Jan. 15, 1903, p. 55.

[‡] For data on weights, see article by W. E. McClintock in Journal Association Engineering Societies, Vol. XI., p. 424.

[§] See table, p. 163.

Numbers are used to designate the sizes of stone on road construction, and stone bought from a crusher is likely to be sold in this way. In such cases it must be borne in mind that these numbers are of local significance. Some plants call their finest product, including dust, No. 1 stone, while others commence to number from their coarsest size or tailings.

CHAPTER XVI

MIXING CONCRETE

The method employed for mixing concrete is immaterial, provided the result is a homogeneous mass of the required uniform consistency, containing the various aggregates and cement in proper proportions. If the color of the mass is not absolutely uniform, that is, if uncoated particles of sand or stone are visible, if masses of stones are separate from the mortar, or if some portions of the mortar are dryer than others, the mixing has not been thorough.

Hand vs. Machine Mixing. First-class concrete may be produced, with careful superintendence, by either hand or machine-mixing.

The relative cost of the two methods depends entirely upon circumstances, and must be estimated for each individual case. If the job is a small one, so that the cost of erecting the plant plus the interest and depreciation, divided by the number of cubic yards to be made, is a large item, or if frequent moving is required, concrete may be and often is mixed cheaper by hand than by machinery. The information which follows concerning both methods will serve as a guide for comparison in special cases.

MIXING CONCRETE BY HAND

The methods employed by different engineers and contractors for handling the materials and arranging the men are nearly as varied with hand-mixed as with machine-mixed concrete. Concrete mixing is seemingly so simple an operation that it is often neglected by the inspector, and poor workmanship escapes detection.

The inspector should lay the greatest stress upon (*a*) exact measurement of the gravel or broken stone, (*b*) thorough mixture of the cement and sand, (*c*) thorough mixture of the mass, and (*d*) care in dumping the concrete into place. The quantity of water used in the mixing and the proper ramming or puddling of the concrete in place are equally important but are less likely to be overlooked.

In proportioning the ingredients, it is poor economy to make allowance for insufficient mixing or improper handling of the materials. The additional cement will be much more expensive than the extra time expended by laborers in securing a homogeneous mixture.

In the first place the mixing platform should be located as near the work

as possible, and so situated that the coarse materials can be conveniently dumped on one side of it and the sand on the other. It should be not less than 15 to 20 feet square if all the work is to be done upon it, and except for a very small job should be of 2-inch plank, planed one side, spiked to, say, 2 by 4-inch stringers about 5 feet apart, so that it can be moved from place to place as required. A 2 by 3-inch strip around the edge will prevent loss of material. If the sand and cement are made into a mortar before mixing with the stone, the platform may be narrower and a mortar box employed in addition.

Methods of Measuring Material. Cement should invariably be measured by weight. In practice this is accomplished not by weighing on scales but by counting packages, since bags or barrels of cement have standard weights.*

The volumes of sand and stone or other aggregate should be distinctly stated in the proportions in terms of the number of cubic feet of each material to a barrel of cement, or else by parts, coupled with the explanation that one part, or barrel, represents a definite volume, such as 3.8 cubic feet. In specifications where the proportions are given by parts with no unit of measurement, the contractor undoubtedly has the legal right to base the volumes of aggregate on the loose measurement of cement, hence the necessity of exact statement of units, as prescribed on page 217.

The sand measure preferred by the authors is a bottomless box similar to the gravel box shown in Fig. 5, page 18, having a depth of about 6 inches, and other dimensions determined by the required volume. The filling of cement barrels or half-barrels with sand is a slower and less accurate process. If the sand cannot be conveniently unloaded close to the measuring platform, it may be measured in a barrow or other wheeled vehicle so constructed that it can be accurately leveled off after filling. For rough measurement ordinary contractors' barrows, whose approximate "large" capacities are given on page 9, are suitable. If more exact quantities are required, however, it takes only a few more seconds to dump the sand from the barrows into a bottomless box.

For gravel or broken stone a bottomless box about 8 or 9 inches deep, shown in Fig. 5, page 18, is a convenient measure. Special barrows built to exact dimensions are more exact measures than ordinary contractors' barrows and, in some cases, than the bottomless box, because an unscrupulous contractor can more easily heap the material in the latter when the inspector's back is turned. Cement barrels are accurate measures, but time is wasted in lifting the shovels when filling, and in dumping them.

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*See page 2.

A measuring barrow car,* built so that it can be handled with a derrick, is sometimes convenient.

Hand Mixing. A detailed description of one of the best ways to mix concrete by hand is given in Chapter II for the benefit of those not familiar with concreting. It is the general opinion of concrete experts that the particular order adopted for mixing the materials has little effect upon the strength of the concrete, provided the materials are turned a sufficient number of times to incorporate them thoroughly. Some engineers prefer to make the cement and sand into a mortar, while others do not add the water until the final turning. The authors have seen excellent work produced by both methods, but prefer the latter chiefly because shoveling the mortar on to the stone involves more labor than handling the dry mixed cement and sand; in fact, comparative tests show that it costs less to mix the cement and sand dry, shovel the mixture on to the stone and mix three times, than to make a mortar, shovel it on to the stone and mix only twice.

Methods variously employed, the first of which is described in detail on page 21, are outlined as follows:

(1) Cement and sand mixed dry and shoveled on to the stone or gravel, leveled off, and wet as the mass is turned.

(2) Cement and sand mixed dry, and the stone or gravel dumped on top of it, leveled off, and wet as the mass is turned.

(3) Cement and sand mixed with water into a mortar which is shoveled on to the gravel or stone, and the mass turned with shovels.

(4) Cement and sand mixed with water into a mortar, the gravel or stone spread on top of it, and the mass turned with shovels.

(5) Gravel or stone, sand, and cement, spread in successive layers, mixed slightly and shoveled into a circle or crater, water poured into the center, and the mass mixed with shovels and hoes.

The last method is applicable only where a small amount of concrete is to be mixed on the ground with no mixing platform or mortar box.

Sand and cement must never be mixed up in advance, as lime and sand are often mixed, because the natural moisture which all sands contain will make the cement set and cake.

The systematic arrangement of the men in pairs, as described on page 21, and insistence upon their shoveling from the bottom of the pile and then turning their shovels completely over, are essentials for thoroughly mixed concrete. In the final wet mixing the materials should be turned in this way two or three times.

For wetting the concrete some engineers specify spraying with the hose,

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*See illustration in *Engineering News*, April 23, 1896, p. 268.

but in practice there appears to be no special advantage in this over ordinary galvanized iron buckets, while with the latter the quantity can be gaged more accurately by filling the required number of buckets in advance. Nearly all the water can be poured on the dry materials before commencing to turn, and the remainder used to wet up occasional dry spots.

The quantity of water is regulated according to the appearance of the concrete after placing. In a thin wall the water will rise to the surface through successive layers so that the first batches in a day's work require the most water. Whatever the quantity, it should be thoroughly incorporated with the other ingredients, and the amount which can be thus incorporated may sometimes be taken as the allowable limit in hand-mixing. The best consistency for different classes of concrete is discussed on page 371.

Distribution of Mixing Gang. Whatever the methods of mixing, the chief requisites for economy are such an arrangement of the gang that each man will have definite duties, and that the number of men on one set of operations will perform their work in the same length of time required by another set of men to perform a different operation or set of operations. A gang should be as large as practicable in order to lessen the cost of superintendence and the general expense.

The best plan, where the size of the gang can be regulated to suit, is to give each man a single operation to perform. For example, let one man or set of men wheel and measure all the sand; let another set of men mix the sand and cement; let a third set be continually employed measuring the gravel or stone; a fourth mixing the mass, while one or two of their number supply water; a fifth filling the barrows and wheeling the concrete to place, and still another set leveling the concrete and ramming or puddling.

It is generally economical to have two batches of concrete in preparation at once, although one set of men usually can measure and mix the sand and cement for two mixing gangs. While one batch of concrete is being shoveled to place or wheeled in barrows, the other batch, either in a different location on the same platform or on a separate platform, may be spread and mixed.

The method of handling a small gang is described on page 21. The arrangement of gangs on two-well managed actual jobs is illustrated in the following outline:

(1) Gang on a core wall for a dike where the sand and cement were mixed dry and spread on to the stone, then wet as the mass was turned.

The large mixing platform was located 30 to 50 feet distant from the excavation, and the concrete was handled in wheelbarrows.

One foreman.

One man wheeling sand to measuring box.

Two men, working alternately at the two ends of the mixing platform, opening cement, and mixing sand and cement dry.

Three or four men, working alternately at each end of platform, shoveling gravel into bottomless boxes.

Six men working alternately at each end of platform, mixing concrete (turning it three times).

Two men handling water.

Four men wheeling concrete, each filling his own barrow.

Four men leveling and ramming.

The average quantity of concrete in proportions 1: 2: 5 laid by this gang per day of ten hours was about 65 batches or 47 cubic yards, with a maximum of about 90 batches or 65 cubic yards.

(2) Gang for a 6-inch foundation for a street pavement, where the sand and cement were made into a mortar and spread on to the stone, and where two mixing platforms were used, one on each side of the street, with a mortar box between them.

One foreman.

Two men mixing mortar in one mortar box.

Four men shoveling stone alternately into two measuring boxes.

Four men working alternately on the two mixing platforms, spreading mortar on stone, mixing concrete, and shoveling to place.

Three men leveling and ramming concrete and also assisting to shovel to place.

One man carrying water and doing other odd work.

The total quantity of concrete in proportions 1: 2: 5 laid per day of ten hours averaged from 40 to 46 batches or 29 to 33 cubic yards per day for the gang. The gang was not quite up to the average, for under given conditions they ought to have turned out regularly 34 cubic yards per day of ten hours.

Approximate costs of concrete mixing are discussed on page 25.

MIXING BY MACHINERY

On all large contracts, machinery for mixing concrete is universally replacing hand labor. The economy of this usually is due as much to the appliances introduced for handling the raw materials and the concrete

as to the saving in the actual labor of mixing. Any arrangement which requires the measuring and spreading of materials by shovelers before entering the mixer results simply in saving the process of hand turning of the concrete and the labor of shoveling it into the vehicle, and this saving is partly balanced by the cost of maintaining and operating the mixer. On a small job this last item almost invariably exceeds the saving in hand labor and renders the expense with the machine greater than without it.

The design of the appliances or plant for handling the materials, and to some extent the selection of the type of mixer, depends upon local conditions, the quantity to be mixed per day, and the total volume of concrete. For a large mass of concrete masonry it is evident that it pays to invest a considerable sum in machinery to reduce the number of men and horses, but if for any reason only a small quantity, we will say not over 50 cubic yards, can be deposited in a day, the cost of expensive machinery cuts a very large figure and hand labor is generally cheaper. In estimating the interest on the cost of the plant which must be charged against a cubic yard of concrete, instead of dividing the interest per day by the usual daily output, the interest for the year must be divided by the total amount of concrete to be laid in the year. In other words, allowance must be made for the days when inclement weather prevents work. To find the depreciation, the value of the entire plant when new, minus its value after the job is completed, is divided by the total number of yards of concrete. Some of the other running expenses, such as the wages of the engineman, may continue from day to day whether or not any concrete is being laid.

Concrete Mixers. An effective concrete mixer not only stirs the mass, which may tend to separate the light and heavy particles, but cuts it again and again, and repeatedly transfers the materials from one part of the machine to another, so that in whatever order they are introduced, the product will be homogeneous. Continuous turning alone does not accomplish the result so quickly or thoroughly as the more complicated motions. The appearance of the concrete as it falls from the mixer will often distinguish the better of two machines.

The larger the machine, the more economical it will be, provided the arrangements for supplying it with material and conveying the concrete to the work permit running at full capacity.

Concrete mixers are of two general classes: (1) continuous mixers into which the materials are fed constantly, usually by shovelfuls, and from which the concrete is discharged in a steady stream, and (2) batch mixers, designed to receive at one charge, say, a barrel or a bag of cement with its proportionate volume of sand and stone, and after mixing to discharge it

in one mass. It is impossible to separate these two classes very distinctly because many of the machines are adapted to either continuous or batch mixing.

The authors are opposed, as a rule, to the use of continuous mixers, unless the materials are measured and fed mechanically, because of the difficulty of uniform feeding. When the ingredients are measured out by hand, spread in layers one above another, and then, starting at one edge, are shoveled into the mixer, the proportions of the materials in the resulting concrete are regulated by the thickness of the layers of the different ingredients rather than by the dimensions of the measuring barrels or boxes. If in one portion of the pile the layer of cement is thicker than in another, the resulting concrete will be proportionally richer. With batch mixers all the materials enter the machine at once; the homogeneity of the product depends upon the character and length of time of mixing rather than upon the care exercised by the laborers in feeding, and less inspection is necessary.

The regulation of the water supply in machine-mixing as in hand-mixing is largely a matter of judgment. Even if the materials were all supplied under absolutely uniform conditions, the same volume of water would not produce from each batch a concrete of uniform consistency, because, as the concrete is laid, the water works up through from one layer to the next, so that more water may be necessary early in the morning than later in the day. It is well, nevertheless, to roughly measure the quantity each time, varying the amount from batch to batch as the condition of the materials and the state of the mass require.

The selection of the type of mixer is often governed by local conditions. If, for example, there is to be a large quantity of concrete, and the machinery can be located at one place, a stationary machine, mounted perhaps on timber framework, with derricks, elevators, or belts, to raise the materials, may be economical. On running work, like a conduit or retaining wall, more portable machines are required, while for thin layers, like pavement foundations, if any machine is used it must be very light or easily moved. If stone for the aggregate is to be broken on the spot, a stationary plant may be built, or the stone may be hauled from the crusher bin to the mixer. In some cases the conformation of the ground will permit of dropping the materials into or through the machine by gravity. Frequently the volume of concrete to be laid is limited by the construction of forms, and a machine of small size is sufficient.

Mixers may be classified in three general types:

Rotating mixers.

Paddle mixers.
Gravity mixers.

Rotating or rotary mixers, as they are usually termed, sometimes mix the materials by simply tumbling them in an oblong or cubical box, and in other cases by throwing them against the deflectors, blades, or plows.

The cubical box is one of the simplest forms of rotating mixers, and formerly was used largely on extensive concrete construction, but is now giving place to modified forms which permit more thorough mixing and the inspection of the material during mixing. The cubical box is of steel, generally mounted on a timber frame similar to the plan in Fig. 109, page 366. The shaft for revolving it runs through two opposite corners and consists of a perforated hollow tube which supplies the water. The measured materials are dropped in from above through a hinged door in the side of the mixer, and the machine after some twelve or fifteen revolutions is stopped, the door is opened, and the concrete dropped into carts or cars. When most of the concrete is out the box is revolved once again to empty it more completely. The mixer itself is inexpensive, but the cost of erection and of raising the stone and sand often renders it less economical than more expensive machines. Cube mixers are also made on a frame and geared so that they may rotate while filling and dumping.

The oblong rotating box machine consists essentially of a square tube open at the lower end, and set on an incline, with a hopper at the upper end for receiving the materials. In one pattern the materials are mixed dry by a worm in this hopper. It is a continuous machine, and, as usually arranged, the materials are mixed and spread in layers on a platform above the mixer, and shoveled or tipped into it as evenly as possible.

The rotating mixers illustrated in Figs. 97, 98, and 99, which contain deflectors, or blades, are usually mounted by the manufacturers upon a suitable frame, although in certain cases it is preferable to construct special timber framework, so that materials may be introduced and the concrete taken away more economically. The larger machines of this type are so constructed that the materials can be introduced from derrick buckets, carts, or barrows. The rotating of the drum tumbles the material and also throws it against the mixing blades which cut and throw it from side to side. Most of these machines can be dumped while running either by tilting them or their chutes. In some styles the materials discharge at a level but slightly lower than that at which they enter, while in the case of at least one machine the discharge is even higher than the entrance so that it can be set in a hole in the ground with no staging around it.

A different style of rotary machine is shown in Fig. 100. It consists of

an open revolving pan in which are stationary plows which mix the concrete. The outlet is through trap doors in the bottom.

FIG. 97.—Rotary Mixer. (*See p.* 352.)

FIG. 98.—Rotary Mixer. (*See p.* 352.)

Of the paddle mixers, those adapted to mix a batch at a time can be more surely depended upon to produce good concrete than the continuous machines. Fig. 101 shows a duplex paddle mixer to be placed upon a raised platform and fed by hand wheelbarrows or derrick buckets. The mixing paddles, on two shafts, revolve in opposite directions, and the concrete falls through a trap door in the bottom of the machine into carts, cars, or wheelbarrows, or upon a platform whence it is shoveled to place.

FIG. 99.—Rotary Mixer. (*See p* 352.)

FIG. 100.—Revolving Pan Mixer. (*See p.* 352.)

The continuous paddle mixer (Fig. 102) is often used for a volume of concrete ranging from 75 to 150 cubic yards per day. Care should be taken that the materials are thrown in near enough the upper end to be thoroughly mixed. The water is usually fed near the middle of the machine so that the materials are first partially mixed dry. They may be measured by shovelfuls, by spreading in layers before shoveling into the mixer, or by automatic machinery.

Measuring the materials by shovelfuls would seem at first thought likely to give a poorer quality of concrete than measuring in boxes or barrels, but with a properly trained gang and periodic checking of the number of barrels of cement to a given volume of concrete, fair results may be obtained. At the Charlestown Bridge piers in Boston (see Fig. 106, p. 362), the contractors, by changing off the men who shoveled into the mixer so as to give them light work half the time, turned out (by steady work) concrete at the rate of about 17 cubic yards per hour. Each feeding gang consisted of five men, three shoveling gravel, one shoveling sand, and one shoveling cement, the size of shovels being so arranged that when all worked together the proper proportions were introduced. The two gangs changed off every half-hour.

FIG. 101.—Duplex Paddle Mixer. (*See p.* 354.)

When the materials are measured and spread in layers before shoveling into the mixer, the machine should be below the measuring platform, and two gangs of men employed, one on each side of the machine, so that one batch may be prepared while another is entering the mixer. This seems like a very simple requirement, yet the authors have often seen a single gang measure out the materials on the ground while the machine stood

idle, and then lift them to a height of perhaps 3 or 4 feet, while the mixed concrete fell to the ground to be shoveled into barrows. With such an arrangement, hand-mixing is cheaper than machine-mixing.

FIG. 102.—Paddle Mixer. (*See p.* 355.)

Gravity machines, properly so called, require no power, the materials being mixed by striking obstructions which throw them together in their

descent through the machine. A gravity concrete mixer is illustrated in Gillmore's "Treatise on Limes, Hydraulic Cements and Mortars,"[*] first published in 1863. In this machine the concrete fell into successive hoppers opened and closed by hand-levers.

A well-known modern type of the gravity machine, shown in Fig. 103, may be increased in length from 4 to 10 feet by adding different sections. In falling through the slanting tube the materials are thrown by the deflectors on the sides and the curved back — the deflectors also acting as tables upon which the stones are coated with mortar — against several series of iron rods which mix them violently together. The inventor claims that by this violence the cement is pounded into the fractures and indentations of the sand and stone so as to increase the strength of the concrete produced. The materials generally are measured in layers on a platform above the machine and fed by shovels, but may be fed by a tipping box or by a derrick bucket. In the latter case the mixer becomes practically a batch machine.

FIG. 103.—Gravity Mixer. (*See p.* 357.)

Another gravity mixer is illustrated in Fig. 104. Four cone-shaped hoppers at the top of the machine receive the materials in layers, with the cement at the bottom and the coarsest material at the top. From these, on the opening of gates,

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[*] Page 229.

the mixture falls into a single cone below, and thence at the will of an operator into a still lower cone, whence it drops into the car or other receptacle.

Portable Concrete Mixing Machinery. Nearly all the types of concrete mixers described are made, at least in their smaller sizes, so that they can be readily transported from one part of a job to another. A few of them are adapted for such work as laying a thin foundation for street paving, while the heavier machines are sometimes arranged upon

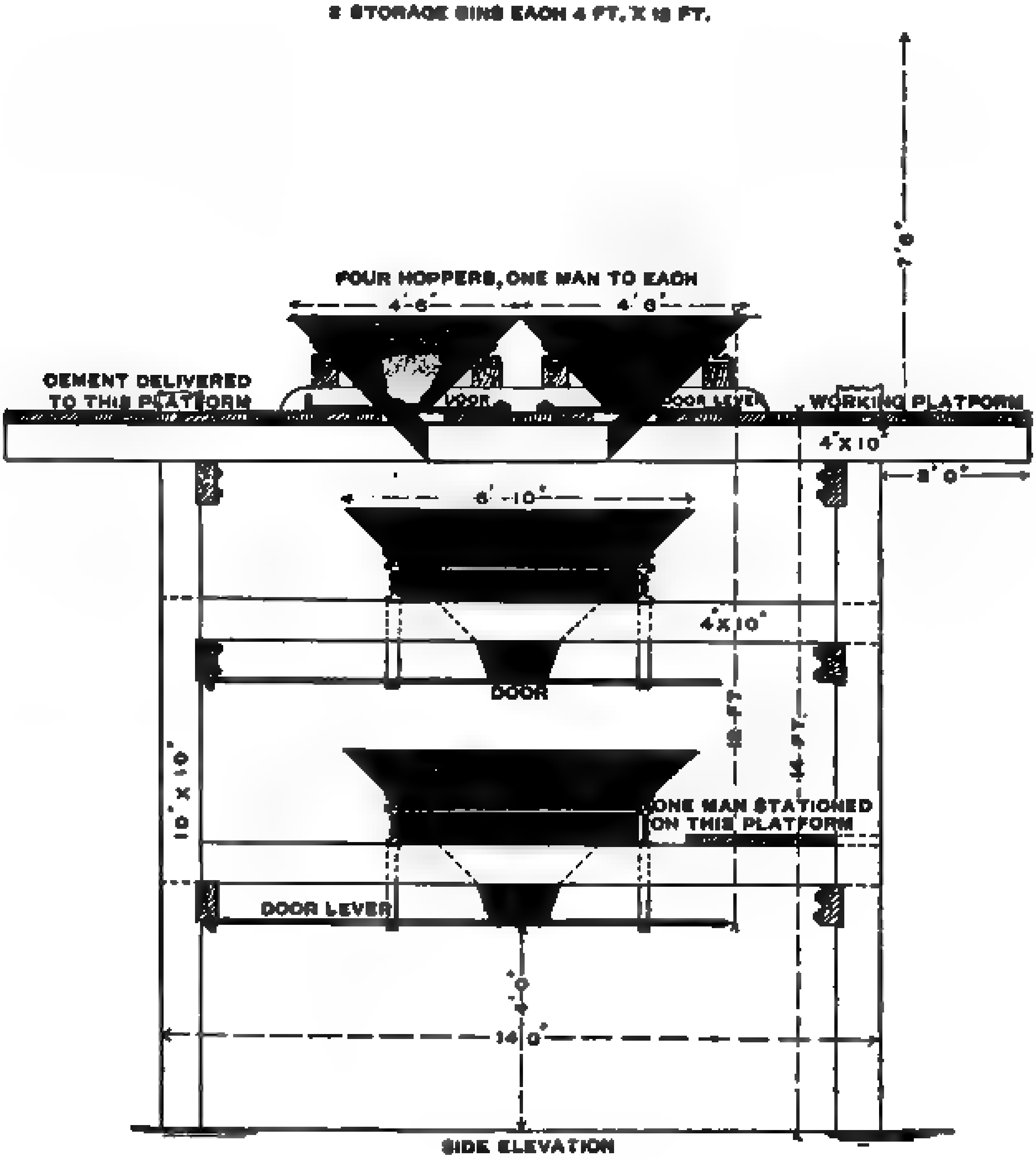

FIG. 104.—Gravity Mixer. (*See p.* 357.)

cars running on a track, so that the concrete can be dropped directly into place from the mixer, or conveyed to place by an endless belt.

On the Chicago & Western Indiana R. R.* a train was made up for preparing and depositing concrete for retaining walls. Three or four cars carried the stone, sand, and cement, and from these the materials were conveyed by wheelbarrows to the mixing car, where the sand and stone were measured, dumped into the mixer, and thence on to a belt conveyor mounted upon a swinging steel boom like a derrick boom, which deposited at any point within derrick swing. The train was hauled by the winding drum on the same engine which operated the mixer, a cable running ahead to an anchor or "dead-man" in the ground.

In building a dam at Chaudiere Falls, P. Q.,† tracks were laid just above and below the site of the dam and parallel to it, and a traveling platform containing the mixer was constructed so as to straddle the dam. The mixer discharged the concrete into the upper end of a tube fitted with a lower telescoping section, so that it could be deposited directly on any part of the dam.

Automatic Measurers for Concrete Materials. The accurate measuring of concrete materials by mechanical means has not been extensively developed. One difficulty, if methods of volumes are employed, lies in the inaccuracy of measuring cement by volume.

One patented device consists of several drums, one for each material, placed directly under the bins containing the cement, sand, and stone, and rotating upon the same horizontal shaft. The quantity of each material is regulated by the position of the gates in the bins and by the speed of rotation.

Another machine delivers the different materials through separate troughs containing Archimedean screws.

FIG. 105.—Measurer for Concrete Materials. (*See p.* 359.)

Another type of measuring machine, the working of which is illustrated in Fig. 105, consists of one or more bottomless storage cylinders, from under which the material flows out on to revolving discs or tables, and is peeled off by stationary adjustable knives which rest upon the discs

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**Engineering News*, Feb. 28, 1901, p. 149.

†*Engineering News*, May 7, 1903, p. 403.

and project into each material a distance determined by the quantity of each required.

A partially automatic measuring arrangement was employed on one section of the Boston Subway, in 1896. Each material fell into a closed chute arranged with gates at such distances apart as to enclose the required volume, whence it dropped into a hopper above the mixer.

Proportioning by Weight. Attention has been called on page 217 to the fact that not only cement, but also sand, stone, and gravel, can be more accurately proportioned by weighing than by volume measurement. When a large amount of concrete is to be mixed, it is possible to arrange apparatus for weighing each material in such a way that less labor will be required than for proportioning by volume. The first cost of the scales may often be more than counterbalanced by the accuracy in proportioning, which permits of leaner mixtures, while at the same time greater uniformity is assured.

In view of these facts, the authors predict that engineers will gradually recognize the advantage of proportioning by weight. In most cases excessive cost may prohibit the use of standard scales, but if the materials are accurately screened and subdivided, the relative weights of each on the same job will be so nearly constant that the weighing can be performed by a simple system of counterweights and levers. With properly constructed gates to the bins it might be possible to arrange for their automatic closing after the required weight of each material had been received in the hopper.

Measurements by weight are employed to excellent advantage by Warren Brothers Company at their various plants where the materials, which consist of stone, sand, and binding material, are prepared for their bituminous macadam pavement. Eight bins containing aggregates of different coarseness drop their materials through gates into a hopper which forms the platform of the scales and is located directly above the mixer. The scale-beam is compound, with as many arms as there are ingredients to be weighed, and each of the arms has a sliding weight and a stop so arranged that the sliding weight can be moved only to the point on the beam which will balance the required weight of one of the materials. When the sliding weights are all at zero and the hopper is empty, the scale balances. The weight on one of the arms is moved out by the laborer who operates the apparatus until it comes to the stop fixed at the point corresponding to the weight of the material to be used from a certain bin. The gate of this bin is opened, and the material allowed to run into the hopper until the scale balances. The weight on the next lever is then slid out, and the

second material deposited in like manner upon the first. When all the materials are thus weighed, the entire mass is dropped into the mixer below.

CONCRETE PLANTS

The design of the plant for handling the raw materials and the concrete usually has more to do with an economical production than the type of the mixing machine. The plant should be drawn or sketched on paper and accurate estimates made of its cost and the expense of operation, so as to determine whether the volume of concrete is sufficiently large to warrant its installation. The authors have occasionally seen expensive machinery, which could not be readily transported to another job, installed on a section of work where, because of the small total volume of concrete and on account of its distribution, hand-mixing was really more economical.

It is evident that the arrangement of any plant must be determined by local conditions, such as the contour of the ground, the distance from which the raw materials are transported, and the class of construction. A description of several plants, successful and economical in operation, may afford suggestions for other work. The illustrations are intended to show the arrangement of the gang and conveying machinery rather than the type of mixer.

Platform over Mixer. A common practice with mixers of various types, where the conformation of the ground permits, and where the quantity to be laid does not warrant the introduction of bins or machinery for handling the aggregate, is to locate the platform for measuring materials directly above the mixer. When ready they are shoveled through a hole in the planking into the machine. One gang of men can measure and spread the materials for a batch while another is shoveling it in. If the mixer is run as a batch machine, the materials may be measured directly into a hopper above it.

A Central Plant. The establishment of a central plant from which the mixed concrete may be hauled to various points as required may be economical in some cities or large towns. This plan has been adopted in St. Louis, Mo.,* for concrete, and is employed in many places for tar and asphalt paving. The plant may be located at a gravel bank or stone crusher, or near a railroad siding, permanent machinery provided which will mix the concrete at a much lower cost than could be done by hand-mixing, and the concrete hauled in carts to the work at but slightly higher cost than the hauling of the dry materials. Most Portland cement con-

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**Engineering News*, March 10, 1904, p. 231.

FIG. 106.—Depositing Concrete of Draw Foundation Pier, Charlestown Bridge. (*See p.* 361.)

crete will not be injured (see page 157) if laid within an hour or two after mixing.

Charlestown Bridge Pier. An economical handling of materials and concrete, where the only machinery was the concrete mixer, is shown in Fig. 106, which illustrates the building of the foundation for the draw pier of the Charlestown Bridge, Boston.* The gravel and sand were brought on scows and deposited so near to the mixer as to require only a short throw or wheelbarrow haul, and were then measured by shovelfuls, as described on page 355. Eight wheelbarrow men, in single file, conveyed the concrete from the paddle mixer, which is shown just to the right of the central mast, along the circular run, then on to the turn-table to the chute for depositing it under water. The entire gang consisted of some thirty-five men, and when working steadily they laid at the rate of about 170 cubic yards of concrete in ten hours, which may be considered a maximum output for a machine of this character, the more usual quantity being from 75 to 100 cubic yards per day of ten hours. The method of depositing concrete from the chute is described on page 394.

Harvard Stadium.† At the Harvard Stadium the builders, the Aberthaw Construction Company, erected a movable tower on each side of the site, and the buckets of concrete and the seat slabs‡ were then taken from cars and conveyed by the cable suspended between the towers to the point where they were needed.

Chicopee River Dam. In mixing concrete for a dam across the Chicopee River in Massachusetts, the contractors utilized a portion of the excavation by locating their mixer against a bank and building out over it a covered platform containing the hopper from which the materials could be dropped directly into the mixer. Stone from the excavation was crushed and elevated to storage bins, whence it was hauled by carts holding exactly the quantity required for a batch, and dumped directly into the hopper above the mixer. The sand was measured and wheeled to the hopper in an iron vehicle consisting of a bucket set on two large wheels which dumped into the hopper by rotating on its axis. The cement was emptied on top of the sand. One batch was mixing in the machine while another was being emptied into the hopper, and thus twenty batches could be handled per hour. The concrete was dumped from the mixer into carts which conveyed it to the dam.

Cambridge Electric Light Station. A portable mixing plant em-

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*Sixth Annual Report Boston Transit Commission, 1900.

†See Frontispiece.

‡See page 470.

ployed on the Cambridge (Mass.) Electric Light Station is shown in Fig. 107. The special feature of the arrangement is the framework containing the mixer. This may be taken up by the derrick, which also supplies it with raw materials, and moved in a few minutes to any other position within derrick swing, so that the concrete can be dropped from the mixer close to or directly upon the place where it is required.

East Boston Tunnel. For measuring materials brought in cars to the work, the contractors for one of the entrance sections of the East Boston Tunnel employed a derrick bucket. The stone was first filled in to a height determined by a gage, then the sand was shoveled on top of it and struck off with a different gage, and finally the required number of bags of cement emptied on top of the sand. The bucket was taken by a derrick and dumped into a duplex mixer.

FIG. 107.—Portable Mixing Plant. (*See p.* 363.)

Cambridge Bridge Piers. When the quantity of concrete to be laid warrants the installation of the necessary machinery, economy requires that the stone and sand shall not be handled at all by laborers. If the stone is crushed on the spot, it may be raised to bins above the mixer

by bucket elevators or belt conveyors, while a similar plan for elevating the material may sometimes be advantageously followed where gravel is used. In building the substructure of the Cambridge Bridge, Boston, Mass.,* the concrete plant was located on a pier resting on piles. The gravel for the concrete was dredged from the harbor and dumped from scows into the water close to the pier. An "orange peel" bucket, operated from a dredging machine on a scow, lifted the gravel, and dropped it into a hopper whence it ran by gravity upon the combination inclined screen described on page 334, which separated the sand, pebbles, and the coarse waste material. Bucket elevators raised the sand and pebbles to bins above the mixer, and from the bins, which were V-shaped, the materials fell by gravity into the measuring hoppers. These were arranged in two sets, an essential requirement for maximum output, so that one batch could be measured while another was being dropped into the mixer. The barrels of cement were brought from the cement shed by a horizontal endless chain, opened on the ground under the mixer, and then three barrels, enough for one batch, were raised at one time by a bucket elevator to one of the hoppers over the mixer.

Williamsburg Bridge Pier. A method of measuring the materials in cars was adopted in building one of the anchorages of the East River Bridge, New York. The cement and sand were stored in bins, and fell by gravity into cars whose capacities were equal, respectively, to the volume of stone and sand required for a batch. Between the tracks upon which these cars ran were two holes in the ground into each of which could be lowered a box of sufficient size to hold one batch of the broken stone, sand, and cement. By tipping the measuring car the broken stone was dumped into the box, the sand fell from another car through a trap door, and the cement was dumped in from the bags. After filling, the box was raised by a derrick and dumped into the mixer.

Parsippany Dike. An endless rubber belt furnishes an excellent means for handling concrete raw materials in a stationary plant. The width of the belt should be not less than 18 inches and the slope no greater than about 22°, which corresponds to 2½ feet horizontal to one foot vertical. Idlers for giving the proper V-shape to the belt are shown in Fig. 108, page 367.

The plan in Fig. 109, page 366, shows the design by Mr. William B. Fuller of a plant used at the Parsippany Dike of the Jersey City Water Supply Co., N. J. The sand was brought to the bins and the stone to

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*For full description see article by Sanford E. Thompson in *Engineering News*, Oct. 17, 1901, p. 282.

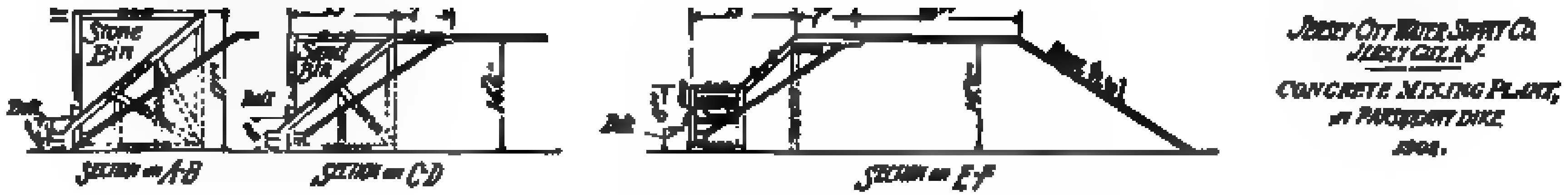

FIG. 109.—Mixing Plant Employing Belt Conveyor. (*See p.* 365.)

the crusher in wagons. A belt conveyor delivered the crushed stone to the bins. At the outlet of each bin a measuring hopper (shown in a detail section, in Fig. 109), containing about 8 cubic feet, received the sand or stone from the bin, and at the ring of a bell the proper quantity of each material for one batch of concrete was dropped upon the conveying belt. The cement was emptied from bags on top of the sand and stone as they were carried past the cement shed. The bin over the mixer had two hoppers. As soon as a batch was delivered to hopper No. 1, the bell was rung again and another batch started into hopper No 2, and while this was filling No. 1 batch was dumped into the mixer.

FIG. 108.—Rollers for Conveyor Belt. (*See p.* 365.)

Blackwell's Island Bridge Piers. At a plant of somewhat similar design built for the piers of the Blackwell's Island Bridge, N. Y., the sand and stone were measured in cars running on a track below the bins, so that they could be moved from one gate to another and discharged at any point through trap doors on to the belt between the rails. The stone was carried up from the crusher by another belt to the top of the bins, where it fell off the belt on to an inclined screen, and rolled into a bin, while the dust, passing through, dropped on to another short belt which carried it to another bin to be used as sand.

CHAPTER XVII

DEPOSITING CONCRETE

The actual handling and placing of the concrete after it has been mixed, and the construction of forms for ordinary mass work, are treated in this chapter. Forms for building construction and conduit construction are illustrated in subsequent chapters on these subjects.

Since the introduction of concrete into engineering construction, the opinions of engineers regarding the best methods of placing it have completely changed. For water-tight work or for the strongest construction it is now recognized that the concrete should resemble as nearly as possible one single solid mass of stone with no joints, and it is the usual practice, although not universal, to specify a "quaking," jelly-like consistency, while many authorities go still further and require water enough to be "mushy" or sloppy. Formerly, for all classes of work, concrete was mixed but slightly more moist than damp earth and laid in alternate blocks 6 to 12 inches thick. Then, after hardening, the forms were removed, and the spaces between filled in.

HANDLING AND TRANSPORTING CONCRETE

In handling and transporting concrete, it is essential to prevent the separation of the stones from the mortar. In hand-mixed concrete, especially for thin walls requiring the stuff to be carried in buckets, there is a tendency to allow the stones to separate on the mixing platform so that a lot of them fall together when cleaning up the last shovelfuls.

With the modern slow-setting cement, and in view of the accepted belief that some time may elapse after mixing without injury to the work, there is less difficulty than formerly in handling the concrete, and it can be readily transported to a considerable distance. Moreover, a wet mixture is much easier to handle, because the stones do not so readily separate from the mass.

The usual vehicle for transporting hand-mixed concrete is a wheelbarrow. For machine-mixed concrete, derricks are suitable if the mass is concentrated near the mixer, otherwise cars running on a track, or in some cases wagons, afford a means of conveyance. A combination of car and derrick work is readily effected by using flat cars with derrick buckets or trays upon them. Galvanized iron buckets are sometimes useful when

building by hand a high, thin wall. A bucket elevator is a poor contrivance for elevating concrete. The mortar sticks to the buckets and the ingredients of the concrete separate as it is thrown from them.

Volume and Weight of Loose Concrete. The volume and weight of loose concrete is of importance in designing the implements or vehicles for transporting it and in estimating the quantities which can be handled under different conditions. The weight of well-proportioned concrete after setting, as stated on page 3, generally ranges from 143 to 155 lb. per cubic foot. When green, it will weigh, after ramming, slightly more than this, say from 150 to 160 lb. The weight per cubic foot loose, that is, in the vehicle which transports it from the mixer to place, depends largely upon the consistency. If mixed very wet, it will settle down to very nearly the volume it has after it is placed, perhaps within 5% of it; but if of dry consistency, the volume of the rammed mass is apt to be as much as 25% less than the loose. A fair average weight of loose concrete may be estimated, then, at about 140 lb. per cubic foot, or 1.9 tons per cubic yard, when mixed wet, and 120 lb. per cubic foot, or 1.6 tons per cubic yard, when mixed dry. The weights and volumes vary, of course, with the proportions used in the mixture and the specific gravity of the stone in the aggregate, but for rough estimates these figures are sufficiently accurate. The volumes of loose mixed concrete required for a cubic yard of rammed concrete, based on the above percentages, are 28 cu. ft. of a very wet mixture and 36 cu. ft. of a dry mixture.

The volume of concrete contained in an iron wheelbarrow load of average size is 1.9 cu. ft. place measurement. A large load is about 2.2 cu. ft. place measurement. Further data is given in Chapter I, page 9.

A single cart on ordinary construction roads will carry about half a batch of concrete of average proportions, which may be assumed as 1 barrel cement to 2½ barrels sand to 5 barrels stone, while with a properly constructed cart which will not overflow or leak, 50% more than this, or about three-quarters of a batch, can be drawn over macadam and paved streets.

DEPOSITING CONCRETE ON LAND

The methods which may be selected for depositing concrete depend largely upon its consistency. If mixed wet, it can be dropped vertically to any depth or passed through an inclined trough or chute. On the other hand, the stones in a dry mixture, that is, of damp earth consistency, will separate from the mortar on the slightest provocation.

To prevent the ingredients separating when passing down an incline, if the mixture is not plastic enough to prevent the stones running away from

the mortar, a pipe with a hopper top and composed of two or more telescoping sections about 15 inches in diameter is often employed. In such a case, the pipe must be often moved or the material shoveled away immediately, to prevent its forming a high cone. Sometimes it is convenient to run the lower end of the pipe into a hopper with a gate at its mouth, so that the concrete may be drawn out into a vehicle, while the pipe and hopper are kept continually full.*

The illustration in Fig. 110 shows at how flat a slope concrete of very wet consistency will run through an open trough. The picture is an actual construction photograph of the Jersey City Water Supply Conduit, and shows the concrete flowing directly from the mixer to the crown of the arch. Mr. William B. Fuller, the engineer, states that when the concrete is mixed of exactly the consistency he likes, it will easily run through an iron trough 15 inches wide by 4 inches deep, set on a slope of 8 feet horizontal to 1 foot vertical.

Fig. 110.—Depositing Concrete through a Trough. (*See p.* 370.)

For water-tight work or for maximum strength the concrete should be

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**Engineering News*, Dec. 25, 1902, p. 537.

placed so as to form a monolith. To do this on a large structure two or three shifts are employed in twenty-four hours, so that no portion of the mass commences to set until fresh concrete has been laid on top of it. In a large reservoir wall at Little Falls, New Jersey, built *en masse* to sustain 40 feet head of water, the only point where the moisture appeared on the surface was at a layer where the work was stopped for one hour at noon. In most structures it is possible to divide the work into sections, each of which is a monolith. Monolithic construction is necessary for columns, beams and floors.

A tipping car for conveying concrete on a track and dumping it into place is shown in Fig. 111.

FIG. 111.—Dumping Car. (*See p.* 371.)

In a thin wall or a structure requiring especial care, such as a tank, it may be advisable to shovel the concrete from the wheelbarrows. Stones which tend to separate can be thus mixed in with the mortar in the wheelbarrow and a very thin layer formed in the molds, so that even if the concrete is mixed very thin the mortar cannot run off from the stones.

CONSISTENCY OF CONCRETE

The terms for specifying the consistency, or degree of plasticity, of freshly mixed concrete are variously used by different engineers. In this

treatise the term *dry mixture* is applied to concrete of the consistency of damp earth, from which the water rises to the surface only after prolonged ramming, and then simply in a glistening film. A *medium* or *quaking mixture* means a tenacious, jelly-like consistency, which shakes on ramming. A *very wet* or *mushy mixture* is one which will not support the weight of a man and into which an ordinary rammer will sink of its own weight; it will run off a shovel unless shoveled very quickly, and will spread out and settle to a level surface after wheeling about 25 feet in a wheelbarrow.

The proper consistency, or wetness, of concrete is a disputed point among engineers, some still holding to the very dry mixture, while others prefer one nearly as liquid as grout. As a result of a series of tests and of practical experience, the authors advocate varying the consistency according to the class of work, and present the following general conclusions:

Medium or quaking concrete is adapted for ordinary mass concrete, such as foundations, heavy walls, large arches, piers, and abutments.

Very wet or mushy concrete is suitable for rubble concrete and for reinforced concrete, such as thin building walls, columns, floors, conduits, and tanks.

Dry concrete may be employed in dry locations for mass foundations which must withstand severe compressive strain within one month after placing, provided it is carefully spread in layers not over 6 inches thick and is thoroughly rammed.

The experiments of the authors show that while dry concrete, very carefully mixed and rammed, is stronger on short time tests, medium mixtures will attain nearly equal strength in six months' time. One of the arguments against very dry mixtures is the difficulty of obtaining a uniform consistency. Occasional batches will invariably be too dry, and it is impossible with ordinary care in placing and ramming to avoid visible voids or pockets of stone which form weak places and allow the penetration of water.

The 1903 specifications of the American Railway Engineering and Maintenance-of-Way Association are as follows:

> The concrete shall be of such consistency that when dumped in place it will not require tamping; it shall be spaded down and tamped sufficiently to level off and will then quake freely like jelly, and be wet enough on top to require the use of rubber boots by the workmen.

A very wet mixture is more suitable for rubble concrete or concrete rubble because the large stones more readily settle into place and bed themselves. In thin walls very wet concrete can be more easily "joggled"

into position so as to conform to the molds and give a smooth surface. The use of a mixture sufficiently wet to flow under and around metal reinforcement has been found by Prof. Charles L. Norton (see p. 428) to be one of the essentials for the preservation of the metal.

Stone pockets may occur even with very wet concrete because of the mortar running away from the stones. This may appear an imaginary danger to many users of concrete who have never employed a very wet consistency, but the authors have seen concrete mixed with too much water, which after setting and the removal of the forms had the appearance of being mixed too dry. In their opinion, however, the limit of wetness for many classes of work is not reached until there is so much water that with ordinary care in hand-mixing it cannot be made to incorporate with the other materials.

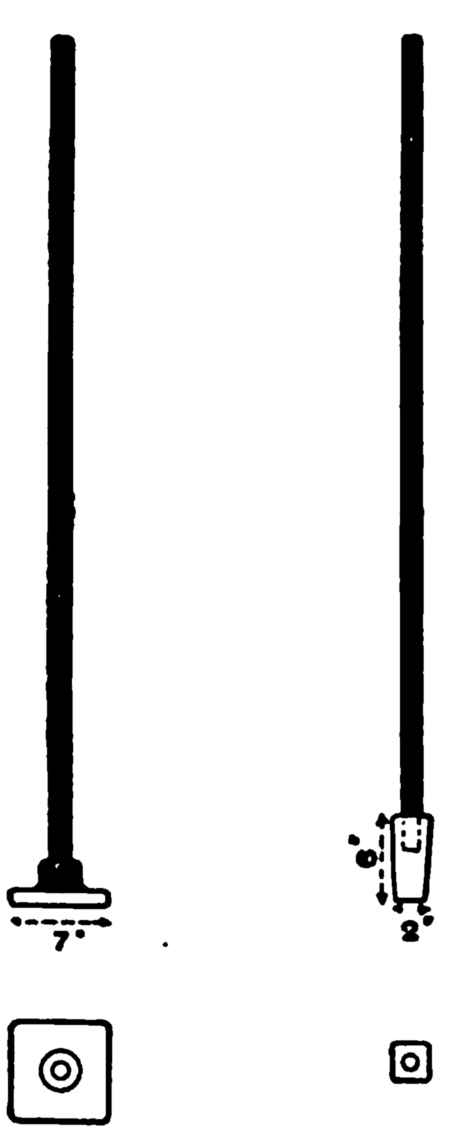

Fig. 112.—Rammers for Dry Concrete. (*See p.* 373.)

RAMMING OR PUDDLING

The method of compacting the concrete or forcing out the air after placing, and the kind of tools to employ for this, depend upon the consistency of the material.

In concrete mixed with a small amount of water the thickness of layers is usually specified at 6 to 10 inches, the former being the most common, but with a very wet or mushy concrete 12 to 15 inches may be placed at once, the chief object being to expel bubbles of air by puddling or joggling. In using very wet concrete there is danger of too much ramming, which results in wedging the stones together and forcing the finer material, the sand and cement, to the surface.

The style of rammers ordinarily used for dry mixed or medium concrete are similar to the forms shown in Fig. 112. The style on the left of the figure is the ordinary type, and on the right is a style convenient for use close to the forms.

The rammer shown in Fig. 113, page 374, which weighs about 8 pounds, is the design of Mr. William B. Fuller for very wet or mushy concrete. The handle may be lengthened, as shown, by screwing a pipe coupling on to the wood.

A "post-hole" tamping bar with iron shoe, shown in Fig. 114, has been successfully used by the authors for mushy concrete. A piece of 2 by

3-inch studding cut to the required length and smoothed off so as to be readily grasped by the hands is also a serviceable tool.

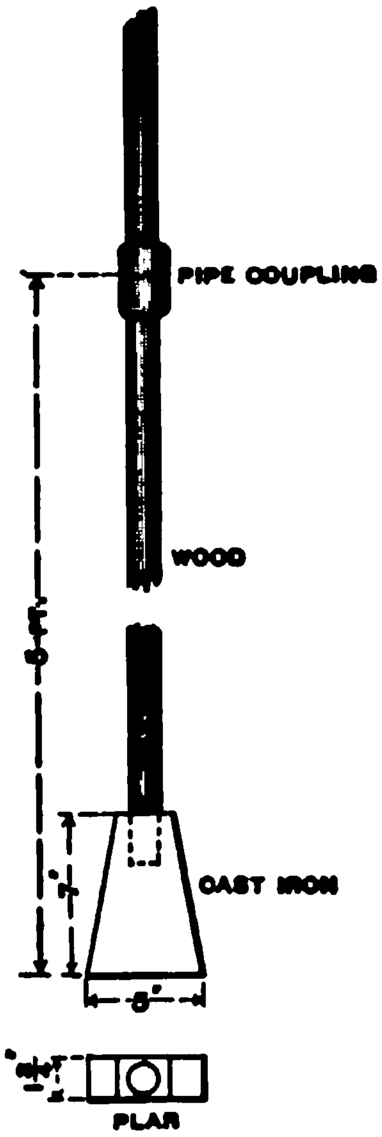

FIG. 113.—Rammer for Mushy Concrete. (*See p.* 373.)

A pneumatic rammer built on the principle of a pneumatic riveting machine, as illustrated in Fig. 115, has been used upon dry mixed concrete with fair success.

Mr. Rafter and Mr. Daniel F. Fulton have designed a rammer based on the principle of the steam drill which is arranged upon a traveling carriage resting upon cross girders which run on tracks. A speed of from 400 to 600 strokes per minute may be maintained with from 4 to 5 horse-power. For ramming street pavements, it should cover 600 to 800 linear feet of a street 30 to 40 feet wide.

Mr. Clarence R. Neher, an advocate of wet concrete, replies to an inquiry of the authors in regard to rammers, as follows:

> I am governed so much by conditions that I use no standard tool, the principle being to use a wedge-shaped rammer of some kind. For the face of the work nothing appears much better than a common spade. This is useful in pushing back stones that have separated from the mass, and also can be used to select the softer and finer portions of the mass and place at the face, while working the spade up and down along the face until it is thoroughly filled. Care must be taken not to pry with the spade, as it will spring the form outward unless excessively strong.
>
> In narrow forms where a man cannot stand in the concrete, a piece of 2-inch by 3-inch scantling, — with the upper portion rounded to make a convenient grip and the tamping end wedge-shaped, — of a length determined by the depth of the form, is convenient and cheap.
>
> In heavy mass work I prefer this same form of rammer to the ordinary type, and thoroughly incorporate the different deposits together, avoiding as much as possible a smooth, flat finish, so frequently insisted on. I consider the use of the term "layers" as describing just what you do not want. I deposit as much concrete in a form as the rammer will penetrate and enter into the deposit below. The

FIG. 114.—Rammer for Mushy Concrete. (*See. p.* 373.)

amount will thus be governed by the size of the form and method of filling.

In elevator foundations we have filled columns 3 feet by 11 feet by 22 feet high in five hours, dumping 14 cubic feet at a time, and not trimming, but shoving the rammer through the mass. The work is absolutely free from voids.

Labor of Ramming. The number of men required for leveling and ramming concrete depends upon the thickness of the wall and the consistency of the mass.

In the table of concrete data in Chapter I, page 9, we have specified 11 cubic yards as the work of an average man in ten hours, including both leveling the material as it is dumped from barrows and the actual ramming. This figure is based upon actual records of a large number of jobs where the concrete was laid of the medium consistency most commonly employed in ordinary mass work. Similarly, a large day's work is placed at 16 cubic yards. Mr. George W. Rafter writes the authors that 4 cubic yards is about an average day's work for an Italian laborer on dry mixed concrete. Mr. Neher estimates for ordinary conditions 10 to 15 cubic yards of wet concrete per man per day with an average of about 12 cubic yards per ten-hour day. Mr. Fuller, who employs a still wetter mixture, considers 25 to 50 cubic yards a day's work for a man joggling.

FIG. 115.—Pneumatic Concrete Rammer. (*See p.* 374.)

On the author's basis of 11 cubic yards per day, the average cost of leveling and ramming mass concrete with labor at $1.50 per day, allowing for superintendence and contractor's profit, is about 18 cents per cubic yard. For a 4 or 6-inch wall the cost may be two or three times this figure.

BONDING OLD AND NEW CONCRETE

In a foundation or other structure where the strain is chiefly compressive, the surface of the concrete laid on the previous day should be cleaned and wet, but no other precaution is necessary. Joints in walls or in other locations liable to tensile stress are coated with mortar, which should be richer in cement than the mortar in the concrete, proportions 1:2 being commonly used.

Some engineers spread the cement dry upon the wetted surface of the old concrete, while others make it into a mortar; the latter method is necessary in many cases to seal the joints between the top of the old concrete and the bottom of the raised forms.

The adhesive strength of cement or concrete is much less than its cohesive strength, hence in building thin walls for a tank or other work which must be water-tight, the only sure method is to lay the structure as a monolith, that is, without joints. If the wall is to withstand water pressure and cannot be built as a monolith, both horizontal and vertical joints must be first thoroughly cleaned of all dirt and "laitance" or powdery scum, wet, and then covered with a very thin layer of either neat cement or 1:1 mortar, according to the nature of the work. As an added precaution, one or more square or V-shaped sticks of timber, say 4 or 6 inches on an edge, may be imbedded in the surface, or placed vertically at the end of a section, of the last mass of concrete laid each day. In some instances large stones have been partially imbedded in the mass at night for doweling the new work next day.

In the New York Subway, work was commenced with no provision for bonding horizontal layers, but it was soon found that more or less seepage occurred, and in one case where a large arch was torn down the division line between two days' work was distinctly seen. Accordingly, at the end of each day's concreting a tongue-and-grooved joint was formed by a piece of timber 4 inches square partly imbedded in the top layer. This was removed before resuming work.

Roughening the surface after ramming or before placing the new layer will aid in bonding the old and new concrete.

CONTRACTION JOINTS

Temperature changes are apt to produce contraction in concrete in air because in temperate climates most concrete is laid during the warm season. Moreover, it is generally recognized that while setting and hardening in air, concretes and mortars contract for a period.

It is probable that this contraction may be due, in part at least, to the

cooling of the cement, which when setting attains a high temperature.* This is further evidenced by the fact that cracks in a thin building wall, 4 or 6 inches thick, open up within a few weeks after being placed, while heavier walls may not crack for several months. The concrete in the interior of a mass like a large dam cools very slowly, and records at the Boonton, N. J., dam indicate that the contraction cracks continue to increase in width for several years. The interior of a large mass like this is but slightly affected by atmospheric changes, and the cracks are but slightly wider in winter than in summer. In an ordinary wall, if no cracks occur after nine months' setting there is no further danger, although after joints once form they will vary in width with the variations in temperature.

Contraction in concrete walls is provided for by forming joints at intervals to divide the wall into separate sections, and confine the cracks to straight lines, or else by reinforcing with sufficient steel to withstand shrinkage.

Joints in vertical walls may be made simply by placing a temporary dam between the molds to remain until the concrete has set, when it is removed and the next section is filled in. In a reinforced wall rods may be run through holes in the dam if it is desired to tie the two sections together. If the old work has thoroughly set and the rods project only a few inches into the new, the adhesion between the old and new work will be so slight that a joint which will open as the concrete shrinks will be formed at the desired point. For bonding the two sections, a V-shaped groove may be molded into the part first laid, or alternate courses may be lapped or toothed out. A water-tight joint may be made by leaving a slit about one-half inch wide and filling this with a plastic material, one of the best for this purpose being pure asphalt of medium hardness. Lime dust is sometimes mixed with the asphalt. Another way of forming a joint is to insert two or more thicknesses of tarred paper.

In building the concrete filter tanks at Little Falls, N. J., which are 15 by 24 feet in horizontal area and rest upon concrete girders, the walls of adjoining tanks were laid on different days, and thus kept separate from each other. Contraction is provided for in each tank by sloping the ledges on which its walls rest, so that, in case of contraction, they will slide without cracking.

At the same plant† occasional expansion wells or vertical openings were built the entire height of the 40-foot retaining wall, to confine cracks to

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*See p. 130.

†Transactions American Society of Civil Engineers, Vol. L, p. 406.

these places, and later, in cold weather, when the cracks were furthest open, these wells were filled with concrete.

From practical experience it appears that heavy walls require fewer contraction joints than light ones. In concrete retaining wall construction in Chicago* joints formed every 50 or 60 feet opened up quite noticeably in cold weather. Where the walls were of small cross-section a hair crack appeared half-way between the joints, tending to show that in thin walls joints should be provided about every 30 feet. A very slight reinforcement, such as ¼-inch rods, spaced 18 inches apart, in a 6-inch wall, tends to assist in preventing cracks. The authors have built solid walls of this character over 60 feet in length which showed no cracks, although if joints were left they opened up slightly during hardening.

The Harvard Stadium, 575 feet in net length or 1390 feet measured around the U, which is illustrated in our frontispiece, is an example of the possibility of providing sufficient steel to withstand the contraction due to hardening and temperature changes.

Mr. A. L. Johnson† has attempted a mathematical demonstration of how to prevent contraction. He states that he has built continuous walls 300 feet long and 8 inches thick which have been exposed to the weather on both sides for a year and are in perfect condition. He considers expansion joints unnecessary for any length of wall if properly constructed. His theoretical explanation of this is as follows:

> Continuous walls will crack vertically in lengths such that the weight of the section multiplied by the coefficient of friction on the soil is equal to the tensile strength of the wall. The temperature required to crack the wall in these lengths is that temperature requiring a shrinkage in excess of the ability of the wall to stretch. Now, plain concrete can stretch very little before cracking. But concrete thoroughly reinforced with metal can take a proportionate elongation of .0018 before cracks will be developed.
>
> The maximum shrinkage that would be required could not be due to a fall in temperature of more than 125 degrees. The coefficient of expansion of concrete is .0000055, which for 125 degrees becomes .0007 per unit of length, or less than one-half the ability of the reinforced concrete to stretch. No crack, therefore, could be produced with a fall in temperature of less than 250 degrees, which, of course, would be impossible to realize in practice.
>
> The quantity of metal used should be enough to equal the tensile strength of the concrete at the elastic limit of the metal. Calling the tensile strength of stone concrete 200 pounds per square inch, and the elastic limit of the

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*Prof. W. D. Pence in Journal Western Society of Engineers, Vol. VI, p. 563.

†*Railroad Gazette*, March 13, 1903, p. 184.

steel 55 000* pounds per square inch, the number of square inches of steel required would be $\frac{1}{275}$ of the number of square inches in the wall section.

Prof. William D. Pence, by very careful experiments at Purdue University, in 1899 to 1901,† determined the coefficient of expansion of concrete in air from changes of temperature to be 0.0000055 per each degree Fahrenheit. He experimented with Portland cement concrete mixed in proportions 1:2:4 broken stone and 1:2:4 gravel. The apparatus was designed to give extremely accurate results, and the variation in the coefficient of expansion in the different tests was from 0.0000052 to 0.0000057 per degree Fahrenheit. Two brands of Portland cement were employed, and in the broken stone concrete, two different stones. The average result for the gravel concrete was 0.0000054 per degree Fahrenheit, and for the broken stone concrete 0.0000055 per degree Fahrenheit. Prof. Pence concludes that "the coefficient of expansion of concrete is about 0.0000055 per degree Fahrenheit. (This value is conveniently remembered as five zeros fifty-five.)" The coefficient of expansion of the limestone used in a part of the tests was the same as that of the concrete made from it. Experiments‡ under the direction of Prof. Hallock at Columbia University gave 0.00000561 as coefficient for 1:2 mortar and 0.00000655 for 1:3:5 concrete. Prof. Burr calls attention to the similarity of this to the coefficient of linear thermal expansion of steel which is about 0.0000066 per degree Fahrenheit. This fact is of great practical value to the engineer in the construction of reinforced concrete because it shows that the concrete and steel will be similarly affected by temperature changes.

The effect of hardening upon the volume, although less definitely determined, has been experimented upon by Prof. Bauschinger,§ of Munich, and Prof. George F. Swain,‖ of the Massachusetts Institute of Technology. As a result, the Committee on Cements of the American Society of Civil Engineers in 1887 reached the following conclusions:¶

First. Cement mortars hardening in air diminish in linear dimensions at least to the end of twelve weeks, and in most cases progressively.

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*This value is for high carbon steel. The elastic limit of ordinary mild steel may only be taken at 30 000 pounds.

†"The Coefficient of Expansion of Concrete," Journal Western Society of Engineers, Vol. VI, p. 549; republished in *Engineering News*, Nov. 21, 1901, p. 380.

‡Burr's "Materials of Engineering," 1903, p. 378.

§Transactions American Society of Civil Engineers, Vol. XV, p. 722.

‖Transactions American Society of Civil Engineers, Vol. XVII, p. 213.

¶Transactions American Society of Civil Engineers, Vol. XVII, p. 214.

Second. Cement mortars hardening in water increase in like manner but to a less degree.

Third. The contractions and expansions are greatest in neat cement mortars.

Among further conclusions of the committee given in this report it is stated that experiments show the contraction of neat cement in air at the end of twelve weeks to be from 0.14 to 0.32%, and of 1:1 mortar, 0.08 to 0.17%. Although these values are corroborated by Bauschinger's* experiments on Portland cement mortars, the results of which also indicate nearly the same contraction for leaner mortars as for 1:1, further data upon the action of concrete made of modern Portland cement is required before accepting the figures as applicable to this. Considère† gives 0.03% to 0.05% shrinkage for lean mortars corresponding to a contraction of about ½ inch in a wall 100 feet long. These various conclusions show that cracks in a newly laid concrete wall are due in part to contraction in setting. In fact, it has been noticed that joints open up in new concrete before it has been affected by external temperature.

It must be borne in mind that this action during hardening has nothing to do with the temperature of the atmosphere, and does not vary with it, but is in addition to the effects of temperature changes. It is possible, however, as suggested on page 377, that the shrinkage may be due in part to the cooling down from the heat evolved when the cement sets.

FACING CONCRETE WALLS

Exposed concrete walls should not be plastered. It is a needless expense, and the results in variable climates are unsatisfactory. It is difficult to apply cement mortar uniformly to the face of hardened concrete, and it is apt to crack off and discolor, especially if the concrete behind it is porous enough for the water to penetrate it. For waterproofing walls not exposed to the atmosphere, cement plaster is sometimes serviceable, as described on page 419.

Mortar for patching irregularities and pockets, which will occasionally occur in the best work, and for filling holes, must contain the same proportions of cement and sand as the concrete, or it will set a different color.

The treatment of the face of concrete is determined by the character of the structure. A fair surface, suitable for work which is not exposed to view, and even for sheds or other buildings where the appearance need not be regarded, has been obtained by the authors on 4-inch and 6-inch walls

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*Transactions American Society of Civil Engineers, Vol. XV, p. 722.

†Considère's Reinforced Concrete, 1903, p. 87.

by using merely a very wet mixture of cement, sand and gravel, with care in placing and puddling so that none of the stones, many of which were 2 inches in diameter, collected in pockets against the forms. Such treatment will result in a sandy finish, showing the joints in the forms less than a smoother one.

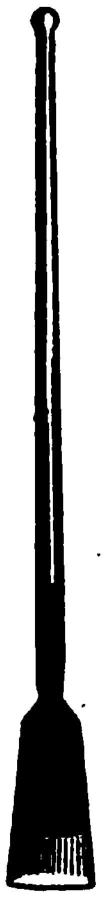

FIG. 116.—Face Cutter. (*See p.* 381.)

To produce a smooth mortar surface, a thin tool like a spade or an ice cutter, shown in Fig. 116, may be thrust down next to the molds as the concrete is placed, so as to force the stones back from the face and allow the mortar to cover every stone, care being taken not to pry the molds.

One of the best methods of finishing for a large smooth surface is to spade or cut the faces as described, and then after the forms are removed to pick them with a hand tool, shown in Fig. 117, or a pneumatic tool adapted for the purpose. The Harvard University Stadium, illustrated in our frontispiece, is finished in this way, and the photograph in Fig. 118 shows a near view of the surface. On the left is the concrete showing the impressions of the plank forms, and on the right is the finished surface. If this picking is performed by hand, it is done by a common laborer. The surface he will cover per day depends upon the hardness of the concrete. It must not be too green or the tool will loosen the stones, while if set very hard the labor is unnecessarily great. On the average, a man may be expected to cover about 50 square feet per day of ten hours. The picks require frequent, at least daily, sharpening. For the best appearance, the size of stone in the concrete should be limited to about ¾ inch to one inch. This method of picking was employed by Mr. E. L. Ransome in the construction of the Pacific Borax Works in New Jersey. A pneumatic tool suitable for this work is made with a circular end containing a number of points, using which a man should cover 400 to 500 square feet per day.

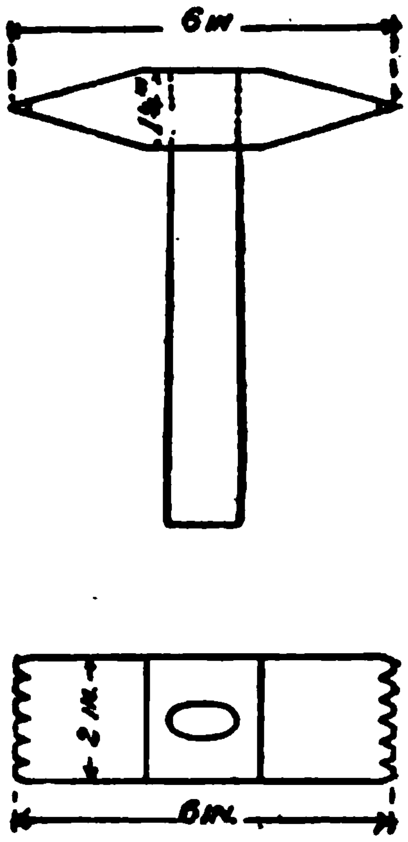

FIG. 117.—Pick for Facing Concrete. (*See p.* 381.)

Mr. C. R. Neher* states that with labor at $1.50 per day bush-hammering will cost less than 1½ cents per square foot.

A surface of washed concrete is shown in the photograph, Fig. 119,

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*Journal Association of Engineering Societies, Jan., 1902, p. 41.

Fig. 118.—Surface of "Picked" Concrete. (*See p.* 381.)

Fig. 119.—Surface of Washed Concrete. (*See p.* 381.)

p. 382. This finish, used by Mr. Henry H. Quimby* for surfacing concrete bridges in Philadelphia, is obtained by hand or with a hose. Hand methods are usually preferable because of the difficulty of applying the hose at exactly the right stage of hardening. In either case the forms must be removed as soon as the concrete is sufficiently hard, — a period varying from 6 hours to 2 or 3 days, according to the character of the cement and the weather, — and the washing done immediately. For washing by hand, a plasterer's float, or a small board 1 by 3 by 6 inches, is used and the cutting is done by sand rolled between the board and the wall, with plenty of water. The concrete face after this process may sometimes be too green for rinsing clean, when the final cleaning is deferred for a few hours. Mr. Quimby states that a laborer should wash and clean 100 square feet of surface in less than one hour. If the concrete has become too hard before washing, a comparatively smooth finish is obtained in a similar manner or by vigorously rubbing the surface with a rough brick.

Mr. H. P. Gillette† mentions a method employed in one case on the New York Central R. R. of chiseling sloping grooves, about ¾ inch deep and 2 inches apart, upon an old discolored concrete surface.

For a very smooth mortar surface, such as may be required for moldings, curved surfaces or carving, the interior surface of the mold may be plastered about ¾ inch thick, by hand or trowel, just in advance of the laying of the concrete, so that the concrete and mortar set up as one mass.

The advocates of dry mixed concrete often require a piece of board corresponding in width to the thickness of the layer of concrete to be placed on edge close to the form, the concrete rammed against it, and then the board removed and the space filled with mortar mixed in proportions 1:2 or 1:3. Another method,‡ adopted by the Illinois Central R. R., which can be used with mortar of a wetter consistency, is to place a thin board or a strip of sheet iron at the required distance from the form, usually about 2 inches, then to fill in the mortar between it and the mold, and the concrete on the other side of it, when it may be removed. Different specifications require thickness of mortar or rich concrete ranging from 1 inch to 6 inches, but in the best modern practice, facing mortar is omitted altogether, and the concrete is made wet enough to present a good surface.§

Marking the surface to resemble masonry is considered unnecessary from an architectural point of view, for the work is actually a monolith and

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*Personal correspondence.

†*Engineering News*, July 24, 1902, p. 66.

‡*Engineering News*, Nov. 29, 1900, p. 380.

§Other methods of facing are described in the Report of the Association of Railway Superintendents of Bridges and Buildings, 1900.

should have that appearance, but if it is desired, triangular pieces may be nailed to the forms, or if tongued-and-grooved plank are used, the horizontal molding may be formed by a strip of wood gotten out to the preferred shape, and planed with a tongue and groove so as to fit between two planks as shown in Fig. 137, page 465.

The size of molding depends upon the class of masonry which is to be imitated. Mr. Edwin Thacher* specifies triangular moldings 2 inches wide by 1 inch deep. Mr. E. L. Ransome has used with good effect a strip like that shown in section in Fig. 120.

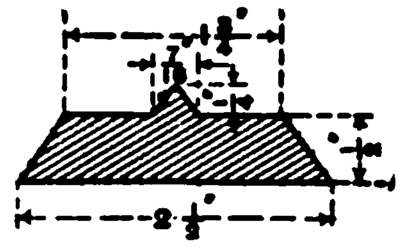

FIG. 120. — Joint Molding. (*See p.* 384.)

The appearance of a concrete face as left by an ordinary form may be smoothed and rendered more impervious and brought to a uniform color by washing with grout. In England† it is customary to use a rather stiff mortar in proportions one Portland cement to three sand, which is applied with a plasterer's hand-float, and worked in so thoroughly as to leave no body on the surface. In the United States the proportions most generally adopted for grout are one part cement to two parts sand, and it is sometimes put on with a whitewash brush or a small whisk broom. 1:2 mortar is considered better for this than richer proportions.

Mr. Clifford Richardson‡ suggests the addition of puzzolanic material, or in other cases, lampblack, to produce uniformity of color.

FORMS FOR MASS CONCRETE

The forms for structures, such as buildings and sewers, are illustrated in the chapters treating upon these subjects.

The best lumber for forms or molds for concrete is white pine because it is easily worked and retains its shape after exposure to the weather. Except, however, where a very fine face is required, motives of economy usually prompt the use of cheaper material, such as spruce or fir, or, for very rough work, even hemlock. Green lumber is preferable to dry because it is less affected by the water in the concrete.

If the planks or boards are thoroughly oiled and are not exposed too long a time to the hot sun and dry air, which tend to warp them, they may be used over and over again. Long exposure, however, will throw the surface out of true, and open up the joints. In some instances the same lumber can be employed in different places. For example, in the con-

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**Cement*, May, 1903, p. 107.

†Sutcliffe's Concrete, 1893, p. 324.

‡*Engineering News*, July 24, 1902, p. 66.

struction of a factory building, Mr. Thompson specified 2-inch tongued-and-grooved roof plank of green spruce for the forms, and after using at least four times, no difficulty was found in laying it on the roof. The planks were merely slightly gritty and discolored by the oil employed to prevent adhesion of cement.

Lumber which is planed one side is essential to a smooth face, and where the forms must be removed within 24 or 48 hours it is sometimes advantageously employed for rough work because the concrete adheres less to planed lumber and that which does stick is easily scraped off, thus effecting a saving of labor which more than balances the cost of planing. Many concrete experts advise the use of beveled edge stuff in preference to tongued and grooved. The edges crush as the board or plank swells, and this prevents buckling.

Square corners and thin projections should be avoided when possible A beveled strip in an external corner will give it a finished appearance.

Either 1-inch boards or 2-inch plank are suitable for forms. The spacing of the studs depends in part upon the consistency of the concrete and the thickness of the walls. If the concrete is laid quite wet and the mass is large, there may be considerable pressure exerted before the cement sets. On the other hand, there is less liability of the boards being forced out of place by ramming than when a drier mixture is used. The authors have found that in comparatively thin walls laid with a wet mixture the stringers may be spaced 5 feet apart for 2-inch plank and 2 feet apart for 1-inch boards. This represents about the limit if an absolutely straight face is desired, and even with this spacing the lumber will spring slightly in places where very short lengths of it are used.

The size of the studding depends upon the height of the wall and the amount of bracing which it is convenient to use. For a low form of 1-inch stuff 2 by 4 inch studs may be satisfactory. If this size is used for a higher wall, horizontal timbers must be placed and carefully braced at distances about 5 feet apart to prevent the studs from springing. For 2-inch plank, as the studding is spaced farther apart, it must be heavier. Common sizes are 4 by 6 inches, 2 by 10 inches, and 4 by 10 inches, depending upon the character of the work and the material at hand. The toes of the diagonal braces which run from the studding down to the ground must rest securely against stout posts or other immovable supports. The use of these diagonals may be avoided in many cases or their number reduced by connecting opposite studs with through bolts or wire. An inexpensive method of connection is shown in Fig. 121, page 386. The wires are wound around opposite studs and then twisted with a stick, as a turn-buckle,

until the studs are the proper distance apart. To remove the forms the wires are cut and then trimmed off close to the concrete.

If in placing the concrete the forms commence to buckle, they must remain in their warped position unless trueness of face is of sufficient importance to warrant tearing down the concrete and replacing it. A carpenter is so accustomed to truing up his lumber after it is in place that it is difficult for him to realize that a thin wall of concrete cannot be straightened in the same way. The fact that a crack once made in concrete which is set is almost impossible to repair cannot be too strongly impressed upon the woodworkers.

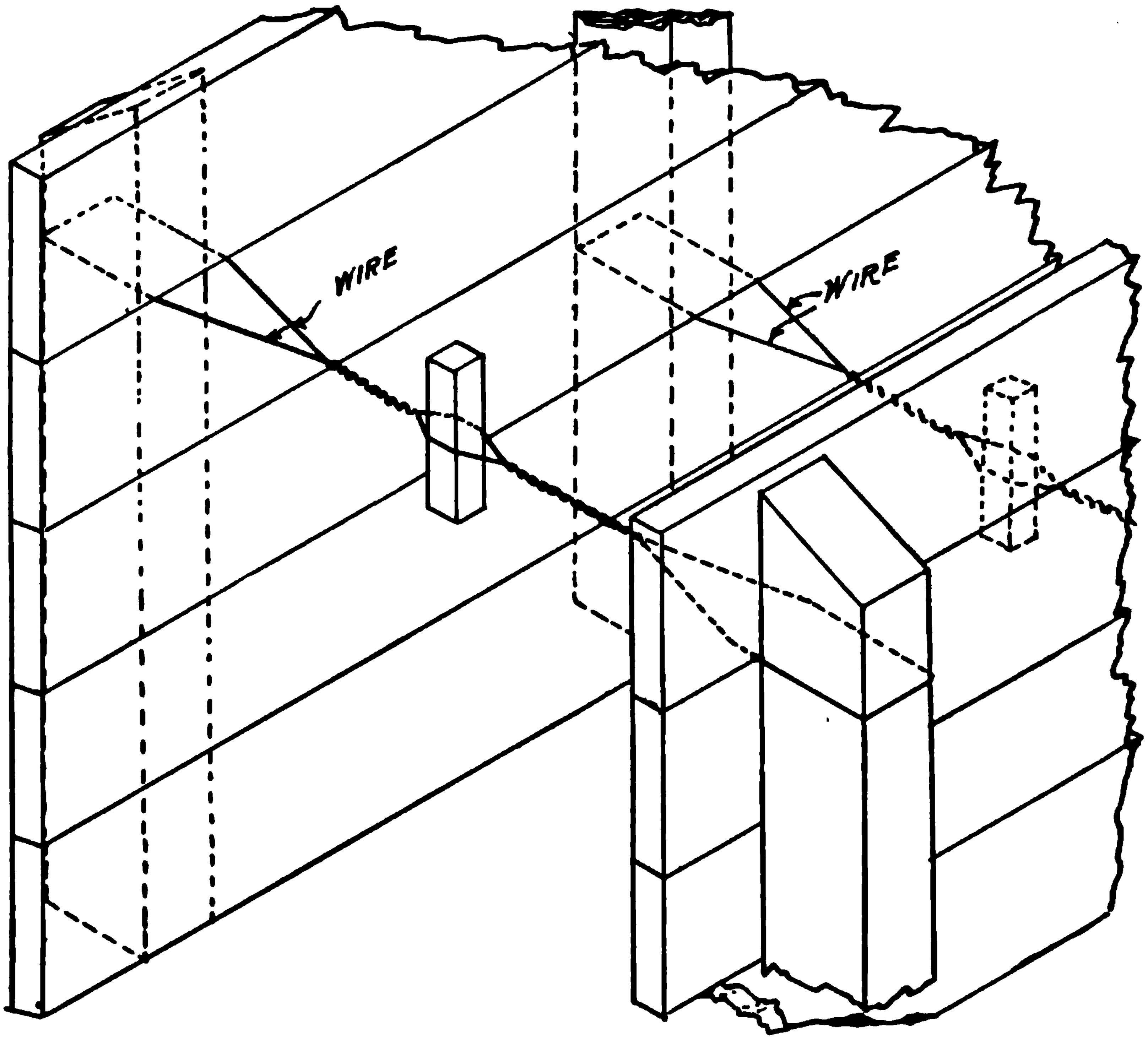

FIG. 121.—Method of Connecting Forms. (*See p.* 385.)

Concrete forms should be nearly water-tight but need not be absolutely so. Cracks of noticeable width which cannot be closed by wetting and swelling the lumber may be battened, and vertical joints between the ends

of planks may be stopped in the same way. Hard soap has also been used for this purpose.*

In a large structure such as a dam, cement bags filled with sand† may be piled to form the temporary end of a layer or series of layers of concrete.

Greasing Forms. Crude oil is an excellent and inexpensive material for greasing forms. This is a petroleum product sufficiently liquid to be readily applied with a large whitewash brush. The object is to fill the pores of the wood rather than to cover it with a film of grease. The oil must be applied every time the forms are set. Thin soft soap or a paste made from soap and water is also occasionally used. On an important job in England‡ the centering boards of arches were covered with strong packing paper soaked with linseed oil.

If the concrete is to set for several weeks before removing the forms, the cohesion of the concrete will be greater than its adhesion to the lumber, and no oil or grease will be necessary, although it is well to thoroughly wet the plank before laying the concrete against it.

Removing Forms. The length of time which concrete must set before removing the forms depends upon the weather, the strain which is to come upon the work, and the consistency employed in mixing.

A good rule to follow when laying wet concrete upon which no pressure is to come immediately is to determine whether it is sufficiently hard by pressing upon it with the broad part of the thumb. If indented, the concrete is too soft to permit of removing the forms. It is sometimes possible in good drying weather, even if slow-setting Portland cement is used, to raise the forms within from 10 to 24 hours after placing the concrete, but care must be exercised that no blow or jar comes upon the fresh work. If the wall is very thin and is to be subjected immediately to earth or water pressure, it may be advisable to allow the forms to remain for several weeks. The setting of concrete is retarded by cold or by wet weather. When mixed very wet, it sets and attains its strength more slowly than when mixed with a small amount of water.

RUBBLE CONCRETE

Rubble concrete includes all classes of concrete in which large stones are placed by hand or by machinery. The term concrete rubble has been applied when the mass consists essentially of large stone laid in joints of concrete instead of mortar.

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*George W. Lee, *Engineering News*, Mar. 19, 1903, p. 246.

†*Engineering News*, Aug. 27, 1903, p. 185.

‡K. Leibbrand in Proceedings Institution of Civil Engineers, Vol. CXIX, p. 227.

Rubble concrete comes in competition with pure concrete on the one hand, and with rubble masonry, — that is, stonework laid in cement mortar, — on the other hand. Its cost in large masses is usually less than that of pure concrete, because the expense of crushing the stones used as rubble is saved, and each large stone replaces a mass of mixed cement and aggregate, thereby saving a portion of the cement. As stone is always heavier than concrete made from the crushed material, because of the pores in the concrete, the replacing of portions of the latter by large stone increases its weight, and therefore its value for certain classes of construction. Large masses of rubble concrete can usually be laid cheaper than ordinary concrete, but where the mass is small and separate machinery or apparatus will be required for handling the large stones, its use may not be advantageous. It is especially suitable where the concrete materials are handled with derricks, because these may be employed to hook the stone or transport it in trays.

In comparison with large masses of rubble masonry laid in cement mortar, rubble concrete of similar quality is almost invariably found to be cheaper because scarcely any skilled labor is required. In a thin wall, not more than 3 feet thick, the rubble masonry may be cheaper because no forms are required. In estimating comparative costs of rubble masonry laid in Natural cement mortar and rubble concrete made with Portland cement, the fact must be considered that a wall of Portland cement rubble concrete may be made thinner than one of Natural cement masonry because it is stronger. The difference in strength is not merely due to the class of cement employed, but to the fact that in rubble concrete the stones are perfectly imbedded instead of being set up on small spawls in the manner customarily employed by stone masons.

The amount of cement used in rubble concrete varies not only with the proportions of the concrete mixture, but with the percentage of rubble introduced. Very much less cement is required in concrete than in a similar quantity of mortar of like strength, but concrete joints must be thicker than mortar joints, so that the result is often more cement is required per cubic yard for concrete than for rubble masonry. However, by employing a large percentage of stone, as was done at Boonton,* the quantity of cement may be brought below that for rubble masonry.

The strength of rubble concrete can be compared only theoretically to that of concrete or rubble masonry, because there are no testing machines in existence of sufficient capacity to break a mass of Portland cement masonry containing large stones. It is generally considered less than that

*See description, page 391.

of plain concrete, but, the authors believe, with insufficient ground. Less cement is contained in a cubic yard, which tends to lessen the strength, but, on the other hand, as stated above, the large stones add density which is a source of strength.

In concrete subjected to tension or bending the introduction of large stones might possibly be a source of weakness by forming planes of adhesion. On the other hand, the stones tooth into the mass and into each other, forming an irregularity of breaking surface which would tend to increase the strength. On long-time tests, too, the strength of the large pieces of stone, which is naturally greater per square inch than the strength of small pieces of broken stone, would naturally come into play. In compression this extra strength of the large stones, especially in their resistance to shearing, has a still greater influence upon the strength of the mass, and besides this they must necessarily bond and wedge with each other.

COMPARATIVE QUANTITIES OF MATERIALS FOR PLAIN AND RUBBLE CONCRETE

The cement and aggregate are often expressed as percentages of the total mass of plain concrete or of rubble concrete. This is confusing because there are various ways of expressing percentages, and, as suggested below, it is therefore clearer in ordinary cases to employ, instead, commercial measurements, such as cubic feet, cubic yards, or pounds.

Before the concrete is mixed, the volumes of materials may be compared by percentages, thus, proportions 1:3:6 have 10% cement, 30% sand, and 60% broken stone; but this is apt to be misleading, since loose volumes, — because of the different voids, — and weights, — because of different specific gravities, — do not exactly correspond to absolute or solid volumes in the finished concrete. By absolute volumes,* for example, a cubic foot of 1:3:6 concrete† may contain 0.079 cu. ft. of solid cement grain, 0.278 cu. ft. of solid sand grains, and 0.491 cu. ft. of solid stone particles, and may be said to have 7.9% cement, 27.8% sand and 49.1% stone. This is an exact method, but such percentages cannot be determined without very complete data.

For comparing costs of different concrete it is therefore best to discard the term percentages, and instead to express the quantity of each material as weights or loose volumes required for a unit volume, — say a cubic yard, — of compacted concrete. By this method a cubic yard of average 1:3:6 concrete (from the table on page 231) contains 1.11 bbl. cement,

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*See example, p. 139.

†See item (23), p. 259.

0.47 cu. yd. loose sand, and 0.94 cu. yd. loose broken stone. If, now, rubble concrete is used and if on the average every cubic yard of this rubble concrete after being laid contains large rubble stone to the amount of 0.3 cubic yards (measured net, as solid stone), we may say that the rubble concrete contains 30% rubble, and each of the other materials are reduced by 30%, thus giving 1.11 × 0.70 = 0.78 bbl. cement, 0.47 × 0.70 = 0.33 cu. yd. sand, and 0.94 × 0.70 = 0.66 cu. yd. broken stone per cubic yard of concrete. From such data, the relative costs of materials for plain and rubble concrete may be readily compared.

Proportion of Rubble in the Mass. The proportion of large stones which can be placed depends upon the size of these stones and upon their distance apart. In a heavy wall or dam the size may be limited simply by the strength of the machinery employed to handle them, whereas in a comparatively thin wall subjected to water pressure, it is evident that the stones should not be large enough to run nearly through the wall and might be limited to one-half or one-third of its width. Larger stones can be used with a wet than with a dry mixture since they bed more readily.

The distance between the stones varies in different specifications from 3 to 18 inches. If the concrete is mixed of dry consistency there must be space enough between the stones to ram the concrete thoroughly and force it into all the recesses, while with a wet mixture the spaces need be regulated merely by the dimensions of the stones in the concrete aggregate, care being exercised that they do not bridge or arch across between the large stones.

The quantity of rubble is usually expressed as a percentage of the total mass of the finished concrete. The percentage may vary from 20% to 64%, both of these quantities being mentioned by Mr. John W. Steven* as used in different places in Scotland. Nearly as much space must be left between two small stones as between two large ones, so that the percentage increases with the size. Into one of the Boonton dikes (4 feet 8 inches thick) of the Jersey City Water Supply Company, — where the stones were hoisted in derrick trays and unloaded by one or two men, — 20% of stone was introduced, and this may be taken as a fair average quantity for concrete containing "one-man" or "two-men" stone. In another Boonton dike, of the same thickness and similar in other respects, the stones were large enough to handle by derricks, and the quantity was increased to 33%, while in the large dam described below, 55% was the average quantity.

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*Proceedings Institution of Civil Engineers, Vol. CXIII.

The amount of rubble may sometimes be most conveniently and accurately measured by weighing it in cart or car-loads.

Methods of Laying Rubble Concrete. The forms for rubble concrete may be built as for ordinary concrete, or the faces of the work may be of cut stone or ashlar masonry.

Ordinarily, derrick buckets are the most suitable apparatus for placing the concrete, because the derrick can also be conveniently used for handling the stone.

One of the best examples of rubble concrete work which has come within the observation of the authors is the dam of the Jersey City Water Supply Company at Boonton, N. J.,* built in 1902-4 under the direction of Mr. William B. Fuller, Resident Engineer. The dam proper contains about 240 000 cubic yards of "cyclopean" or concrete rubble masonry, and the contract price at which this was let, which covered all labor and all materials excepting the cement, was $1.98 per cubic yard. Other bids ranged from $2.20 to $3.60. The rubble stones, which actually averaged in size from 1 to 2½ cubic yards each, were brought from the quarry about three miles distant over a standard gage track built for the purpose, and the stone for the concrete aggregate was also broken at the quarry, although it was not touched by hand from the time it entered the crusher until it was deposited in concrete. One of the distinctive features of the construction was the consistency of the concrete, which was mixed extremely wet, in fact, about like pea soup, so that when dumped it spread out, forming a level bed for the stone. As soon as a bucket of concrete was dumped, a large stone, which had come from the quarry on flat cars, was picked up by one of the stiff-legged derricks ranged on trestles along each face of the dam, and dropped, — with force, not gently lowered, — usually with its smoothest face down, into the mushy mass. Settling into place, it bedded itself in the concrete, and laborers joggled it with crowbars so as to bring it to a firm bearing and drive out all air bubbles. A stone lifted after placing left a bed conforming to the irregularities of the stone, and having the appearance of mortar, no stones being visible. Scraping this mortar in places showed that the stones of the concrete were covered with an exceedingly thin film of mortar.

The labor of actually placing the concrete and stone after bringing them to the dam may be estimated from the fact that each stiff-legged derrick supplied a gang of three or four laborers dumping concrete and joggling the stone, with one foreman mason, who not only looked after the depositing

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*See drawing, Fig. 155, Chap. XXV. See also *Engineering Record*, Aug. 8, 1903, p. 152.

of the stone in the concrete, but also spent some of his time on the face stone masonry. In addition to these, there were the men mixing concrete and handling the cars of stone. Mr. Fuller stated that seven derrick gangs averaged about 700 cubic yards of concrete rubble masonry in ten hours, or about 100 cubic yards to a derrick. A maximum day's work for a derrick was about 125 cubic yards.

The concrete was proportioned 1 part Portland cement, 2¾ parts sand, 6¾ parts broken stone, the latter ranging in size from fine particles up to 3 inches in diameter. The masonry contains about 55% of rubble, the large stones being kept at least far enough apart so that the fist could be thrust between them. About 0.6 barrels of cement were used per cubic yard of concrete rubble masonry. This quantity is less than is generally used in a rubble wall built of fairly well dressed stones laid in 1: 2 cement mortar; and where water-tight rubble is required and the stones are accordingly left as rough as possible, the quantity of cement is apt to average slightly more than one barrel per cubic yard.

In a dam built in eastern Connecticut in 1899 to 1901,* where methods somewhat similar to those just described were employed, the quantity of cement averaged about two-thirds barrels per cubic yard of masonry.

The masonry dry dock at the Charlestown Navy Yard, which was begun in 1900, furnishes an example of rubble laid in dry mixed concrete. The stones, which were placed about 18 inches apart in all directions, averaged about ½ cubic yard in volume, and had comparatively square faces and level beds. They occupied less than one-third of the total volume of the concrete. The concrete, mixed in proportions about 1 part Portland cement to 2 parts sand to 5 parts gravel, was deposited from buckets, and thoroughly rammed, and the stones, after washing with a hose, were placed by derrick. If a stone did not bed itself properly, the derrick picked up a heavy weight and allowed it to drop several times upon the stone to ram it into place.

DEPOSITING CONCRETE UNDER WATER

Although some engineers still specify that no concrete shall be laid under water, the many important structures which have been built of late years upon foundations of concrete deposited loose, to set and harden under water, prove that excellent work can be performed with proper selection of materials and care in laying. It is absolutely necessary, however, to lay the concrete by some means which will prevent the separation of the ingredients as they pass through the water. This has been accom-

*Described by Herbert M. Knight, *Engineering News*, June 12, 1902, p. 470.

plished, as discussed in the succeeding pages, by the following methods: (1) passing the concrete through a tube in a continuous flow, (2) lowering it in large buckets from which the concrete may be dropped in large masses, (3) confining it in bags, (4) forming the concrete into blocks on land, and after setting placing them by machinery or by floats, and (5) allowing the concrete to partially set in air and then depositing it in a "plastic" condition.

For sea water construction, the cement should be carefully tested to see that it is of standard quality.* Occasionally the water of a stream or pond may be impregnated with by-products, such as sulphuric acid from industrial plants, or with mineral impurities which prevent the concrete from setting properly.

Cofferdams, which need not be water-tight, are almost always necessary to prevent the concrete from spreading and the cement from washing away.

Laitance. "Laitance" is a French word, quite generally adopted in the United States and England for the light-colored powdery substance which is held in suspension by the water when cement or concrete is deposited below the surface. On land the same substance forms on the surface of concrete which has been mixed very wet.

The analysis of a sample of laitance† showed its composition to be as follows:

Silica (SiO_2)	16.00%
Alumina and Iron (Al_2O_3, Fe_2O_3)	8.66 "
Lime (CaO)	47.40 "
Magnesia Oxide (MgO)	2.40 "
Ignition loss	23.60 "

If calculated to a water and carbonic acid free basis the analysis becomes:

Silica (SiO_2)	20.94%
Alumina and Iron (Al_2O_3, Fe_2O_3)	11.30 "
Lime (CaO)	62.04 "
Magnesia Oxide (MgO)	3.14 "

Mr. Richardson notes that this composition corresponds with that of a normal Portland cement except that it is unusually high in alumina and iron, a fact which may be explained by the large amount of magma detected in the thin section examined. He further states:

> I have had a thin section ground, but find that it shows no structure which is characteristic. The section consists largely of amorphous material of an isotropic nature, that is to say, it does not affect polarized light. It reveals a considerable amount of a yellow substance which seems to be the

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*Also see Chapter XVIII.

†Analyzed for the authors by Clifford Richardson.

> undecomposed magma contained in the original cement. I have formed a material very similar to the "laitance" by shaking Portland cement with water, decanting the finer portion and allowing it to settle out and harden. This material, like your "laitance," is rather soft, and this is due to the fact that the Portland cement is much more thoroughly decomposed under these conditions than under ordinary ones, and this accounts for its character.

It is evident from these facts that the milky laitance which appears on concrete laid under water represents an actual loss of cement, which should be prevented by confining the mass until it reaches its position.

Depositing Concrete through Chutes. In his Treatise On Limes, Hydraulic Cements and Mortars,* Mr. Gillmore refers to a "trémie" used in laying concrete under water in Chesapeake Bay. This consisted essentially of a tube of boiler iron about 2 feet in diameter, and long enough to reach the place where the concrete is to be deposited. Similar apparatus is still employed for forming layers of concrete under water.

When building the piers of the Charlestown Bridge, Boston, a cofferdam was first constructed, and then a tube, about 14 inches in diameter at the bottom and 11 inches at the neck, with flaring top, was suspended by a differential hoist from a moving platform, as shown in Fig. 106, page 362. The tube was made in removable sections bolted together by outside flanges so that its length could be readily varied. Mr. William Jackson, Chief Engineer for the bridge, describes† the method of operation as follows:

> The foot of the chute was allowed to rest on the bottom, and was filled with concrete dumped from wheelbarrows. The chute was then raised slowly from the bottom, allowing a part of the concrete to run out in a conical heap at the foot, while the loss was made good by dumping in more concrete at the top. The truck bearing the chute was then moved from side to side of the dam, so as to leave a ridge or bank of concrete crosswise of the pier, the chute being kept always filled or nearly filled by dumping more concrete into the hopper. The height of the ridge of concrete was regulated by the height to which the foot of the chute had been raised from the bottom. When the ridge was completed across the dam, the traveller supporting the truck was moved a short distance lengthwise of the pier, and the truck was moved back again across the dam, parallel to its former course, allowing the concrete to run out over the edge of the bank first deposited, widening it on the side to which the traveler had been moved, and this process was continued until the whole area of the foundation was covered with a layer of concrete, upon which, when it was sufficiently hardened, another similar layer or course could be deposited.

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*Page 236.

†Third Annual Report, Boston Transit Commission, 1897, p. 74.

The thickness of each course depended upon the height to which the foot of the chute was raised above the top of the preceding course. Courses were laid up to 6 feet in thickness, but it is thought that the best results were attained with a thickness of 2 or 2½ feet.

If the bank is made too high, or if the bottom (or the top of the preceding course) is very uneven, or if the piles interfere with the motion of the chute, or if the chute is moved along or raised too rapidly, the concrete is likely to run out so fast as to empty the chute entirely before the flow can be checked. In this event the "charge" is said to be "lost," and the chute must be lowered again to the bottom and refilled. When the charge is lost the water rises inside the chute to the same level as that outside, and into this water the concrete must be dumped until the water is wholly displaced or absorbed by the concrete. This has a tendency to wash the concrete, and to separate the cement from the sand and gravel, and as it generally takes a cubic yard or more of concrete to displace all the water in the chute, there is danger that a rather large body of badly washed concrete will be deposited whenever the charge is lost. This danger threatens not only when the charge is accidentally lost, but whenever work is begun in the morning or after the mid-day intermission; for whenever the work stops the charge must be allowed to run out lest it set in the chute.

To obviate partially the evil of washed concrete, the contractor was directed, whenever work was begun after an intermission, or whenever the charge was lost or water leaked into the chute, to throw into it, before each wheelbarrow-load of concrete, until the water was displaced, a quantity of dry cement. He was also directed to begin work after an intermission with the chute near the center line of the pier, so that any body of washed concrete resulting would be completely surrounded by sound concrete.

After the workmen and the inspector had gained experience with the chute, the accidental loss of the charge was not a frequent occurrence, and the danger of an occasional body of partly washed concrete, surrounded as it must be by good concrete, was not looked upon as a very serious matter.

A difficulty sometimes met with in using the chute is that when a sudden rush of concrete takes place, even if the charge is not entirely lost, the concrete within the chute often falls far below the level of the water outside. The outside water then, especially if there is a deficiency of sand in the concrete, is likely to force its way through the concrete remaining in the bottom of the chute, tending to separate the cement from the sand and gravel, and making the concrete too wet, and so threatening a complete loss of charge. If there are any leaks in the joints of the chute, water comes in and tends to cause loss of charge, and this leakage is especially troublesome when the concrete in the chute falls below the level of the water outside.

The chute seems to work best when the concrete is mixed not quite moist enough to be plastic. If it is mixed too wet the charge is likely to be lost; if very dry there is a tendency to choking of the chute. The working of the chute is affected also by variations in the proportions of sand and

gravel. With gravel in excess the outside water too readily forces its way in at the bottom. With an excess of sand the concrete tends to clog in the chute.

Sometimes when the concrete becomes clogged in the upper part of the chute, the concrete below the clogged place continues to flow out, leaving a vacant space into which water forces itself through the concrete remaining in the bottom of the chute. When the clogged concrete above is loosened, it falls into this body of water, which, unable to find exit by the way through which it entered, is displaced by the falling concrete, and rises into the hopper, sometimes to a level considerably above that of the water outside.

In the construction of the foundations for the piers for the Cambridge, (Mass.) Bridge,* a tube was used in much the same way as that employed for the Charlestown Bridge. The concrete was dumped from derrick buckets into a hopper, below which was a tube 16 inches in diameter at the top and 22 inches in diameter at the bottom, built in 4-foot cylindrical sections, which telescoped one another, so that a length varying from 4 to 40 feet could be obtained. Each layer of concrete was 1 to 2 feet thick. The tube was suspended from a traveler running upon a pair of traveling trusses which rested at each end upon tracks laid on top of the cofferdam, so that concrete could be deposited at any point within the rectangle.

Depositing Concrete from Buckets. The opinion of engineers is divided as to whether the best method of depositing concrete under water is by a chute, as has just been described, or from a bucket. The objection to the former is the difficulty in always maintaining a continuous flow, while with the latter it is not so easy to place the layers uniformly and to prevent the formation of mounds which are more or less washed by the water. With careful superintendence, however, either of these methods is satisfactory.

The best results can be attained with buckets so constructed that the material flows out through the bottom. A mass of concrete deposited under water must be disturbed as little as possible, and in tipping a bucket the material is apt to be stirred. Various buckets with bottom doors have been devised for opening automatically when the place for depositing is reached. In one type, used in 1900 at the Charlestown Navy Yard, the slackening of the rope released latches which fastened the trap doors so that they opened as soon as the bucket commenced to ascend. Another style, designed by Mr. John F. O'Rourke, is shown in Fig. 122. The photograph shows the bucket closed. When it reaches the bottom the

*See article by Sanford E. Thompson, *Engineering News*, Oct. 17, 1901, p. 282.

handle slides down, allowing the doors to swing open and the concrete to drop out in a single mass. The bail catches when it has dropped to the bottom, so that when hoisting the bucket the doors remain open. Covers

FIG. 122.—Bucket for Depositing Concrete. (*See p.* 396.)

prevent the water from rushing in at the top as the bucket is being lowered, and the V-shaped bottom lessens the disturbance of the water.

Depositing Concrete in Bags. Bags, varying from small paper or mus-

lin bags to jute sacks containing 100 tons,* have been employed in the past for holding concrete together as it passed through the water. In some cases the concrete has been placed in the bags dry.†

Mr. John Willet,‡ in building the breakwater at Fraserburgh Harbor, Eng., employed bags holding from 28 to 50 tons of concrete. A bag was placed in the hopper bottom of a barge filled with concrete, and sewed up as the barge was being warped to place. When the doors of the hopper were released it fell into place.

John C. Goodridge's method§ of laying concrete under water, employed in 1887, consisted in enclosing the concrete "in paper bags or other soluble envelopes, and then lodging the bags or envelopes so filled in the desired position under water, in such a manner that the bag or envelope shall not be ruptured until after or at the time it and its contents are in place."

Molded Blocks. Under some conditions, especially where it is difficult to construct a cofferdam and monolithic work is not required, blocks of concrete of any desired shapes are molded on land and placed after setting.

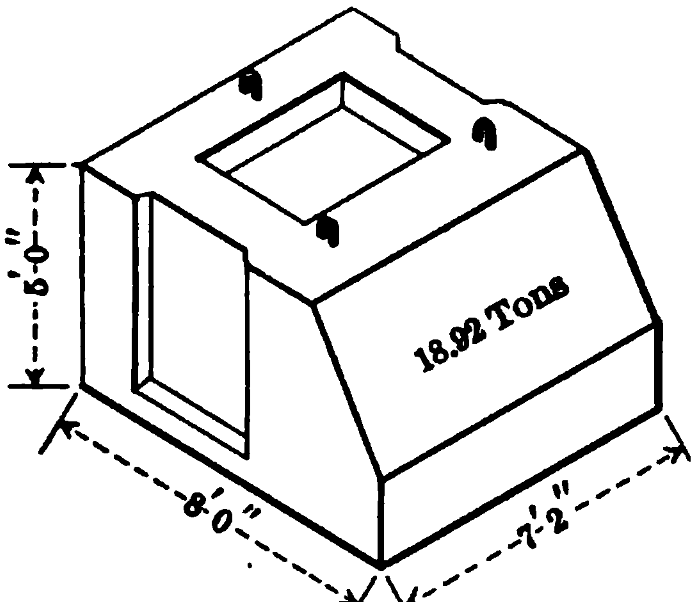

FIG. 123.—Face Blocks of Buffalo Breakwater. (*See p.* 398.)

On the Buffalo breakwater,|| blocks weighing from 15 to 20 tons, one style of which is illustrated in Fig. 123, were employed in parts of the structure. For handling them, three iron bolts having legs bent to an angle at the ends and of unequal length,—one 24 inches long and the other 12 inches long,—so that the strain would occur in two separate planes, were sunk into the top face of each block. After placing them in position, grooves molded into their adjacent faces were filled with concrete so as to dowel them together.

In the harbor of the Welland Canal, Ontario,¶ blocks of somewhat smaller size were used just at the water level, with mass concrete placed on top of them. For handling these blocks four vertical channels, two on each side, were molded into each block, with recesses just below the central points to catch the four hooks used for moving it. As the hooks passed

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*A. E. Carey in Proceedings Institution of Civil Engineers, Vol. LXXXVII, p. 101.

†Lt. Col. J. A. Smith, *Engineering Record*, March 23, 1895.

‡Proceedings Institution of Civil Engineers, Vol. LXXXVII, p. 124.

§U. S. Patent, No. 358 853.

||See article by Major T. W. Symonds, *Engineering News*, May 29, 1902, p. 426.

¶*Engineering News*, May 15, 1902, p. 382.

down in the channels, they projected so slightly that a block could be set close to the last one placed, and the hook removed without disturbing it.

As early as 1873, concrete blocks ranging in size from 13 to 60 tons in weight were used by the Department of Docks in New York City,* and in 1900 this method of construction was still in operation in that city.

In Belgium in 1899, for breakwater construction,† blocks about 25 feet square and 82 feet long, weighing 3 000 tons, were formed by building on the shore metal caissons of the required size, lining them with concrete, then floating to place, and removing plugs in the bottom so as to allow them to sink. The remainder of the concrete to fill the caisson was deposited in the interior.

Depositing Dry Concrete under Water. By dry concrete is meant in this case a mixture of aggregates and cement without water. This method, although occasionally practised, is undoubtedly one of the worst to employ in laying concrete under water. No matter how carefully the concrete is placed, more or less of the cement is carried off by the water. Experiments by Mr. B. B. Stoney‡ show, as one would expect, that a wall laid in this way is honeycombed, and is not nearly so dense as that formed of concrete mixed with water in the usual way before placing.

Plastic Concrete. Plastic or, as it is termed by Mr. Faija, "reset" concrete was once employed in England.§ The concrete was mixed on land with the smallest possible quantity of water, and allowed to set there about three to five hours, or until it attained the consistency of wet clay, before being deposited in the water. Mr. Kinniple claimed that setting eight hours on land before placing did not reduce the ultimate strength of the concrete, and that less of the cement was washed away.

*"Fabrication of Beton Blocks by Manual Labor," by Schuyler Hamilton, Transactions American Society of Civil Engineers, Vol. IV, p. 93.

†See paper by L. Vernon Harcourt in Proceedings Institution of Civil Engineers, Vol. CXII, p. 2.

‡Proceedings Institution of Civil Engineers, Vol. LXXXVII, p. 230.

§W. R. Kinniple, Proceedings Institution of Civil Engineers, Vol. LXXXVII, p. 65.

CHAPTER XVIII

EFFECT OF SEA WATER UPON CONCRETE AND MORTAR*

By R. Feret,
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The principal conclusions which have been reached by the author of this chapter, as discussed in the following pages, may be summarized as follows:

(1) No cement or other hydraulic product has yet been found which presents absolute security against the decomposing action of sea water. (See p. 400.)

(2) The most injurious compound of sea water is the acid of the dissolved sulphates, sulphuric acid being the principal agent in the decomposition of cement. (See p. 401.)

(3) Portland cement for sea water should be low in aluminum (see p. 403), and as low as possible in lime. (See p. 402.)

(4) Puzzolanic material is a valuable addition to cement for sea water construction. (See p. 404.)

(5) As little gypsum as possible should be added, for regulating the time of setting, to cements which are to be used in sea water. (See p. 401.)

(6) *Sand containing a large proportion of fine grains must never be used in concrete or mortar for sea water construction.* (See p. 407.)

(7) The proportions of the cement and aggregate for sea water construction must be such as will produce a dense and impervious concrete. (See p. 407.)

EXTERNAL PHENOMENA

At present there is no hydraulic product which is known to be capable of resisting absolutely the decomposing influence of sea water. It is true that some concrete masonry has remained intact for a very long time in salt water, but with our present knowledge it is impossible to say why these structures have resisted so well, and there is little doubt that the cements from which they were made might have decomposed rapidly if they had been used under different conditions. In some cases, on the other hand, similar large structures subject to the action of sea water were

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*The authors are indebted to Mr. Feret for this chapter, which has been especially prepared by him for this Treatise.

ruined in a few years and were torn down and completely rebuilt. Notable instances of this kind are the failures which occurred in the ports of Aberdeen,* Dunkerque, and Ymuiden.

Such occurrences have aroused great interest in the subject of the action of sea water upon mortars, and but few questions have received more careful study. In spite of this, however, it cannot be said that any sure means of preventing these failures have been found.

The decomposition manifests itself in various ways: sometimes the mortar softens, and little by little becomes disintegrated; sometimes the mortar becomes covered with a crust which finally cracks off; more often fine white veins develop on the surface of the mortar, these gradually grow large and open, the mortar swells, cracks, and falls off in small pieces or collapses in a pulp-like mass. Almost always the interior of the decomposed mortar is found to contain a soft white material which may be easily separated from it. The chemical composition of this substance is not, however, constant.† Generally, the more advanced the state of decomposition, the more readily the white material can be extracted from the mortar and the richer it is in magnesia. The proportion of sulphuric acid in it also increases with the degree of decomposition, though less uniformly.

ACTION OF SULPHATE WATERS

For several years the injurious action of sea water upon hydraulic compounds was attributed chiefly to the magnesia in the water. It is noteworthy, however, that chloride of magnesia is almost without action, while sulphate of magnesia acts very energetically upon cement, and it has now been ascertained that magnesia plays only a secondary part, while in fact it is the sulphuric acid combined as a soluble sulphate which is the real cause of the decomposition.

This has been confirmed in practise by the destruction of masonry washed by water which has traversed earth containing gypsum, or built from mortar made with sand which has been extracted from strata containing sulphate of lime.‡ A consideration of this fact makes it apparent how dangerous it is to use, in concrete or masonry subject to the action of sea water, cements to which gypsum has been added for the purpose of regulating the rate of their setting or of increasing their initial strength.§

There are numerous instances in which brick masonry has rapidly de-

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*Smith, Proceedings Institution Civil Engineers, Vol. CVII, 1891–92.

†Feret, Annales des Ponts et Chaussées, 1892, II, p. 93.

‡Bied, Annales des Ponts et Chaussées, 1902, III, p. 95.

§Feret, Annales des Ponts et Chaussées, 1890, I, p. 375.

composed because the bricks, burned with coal, contained alkaline sulphates which when drawn out by water attacked the mortar of the joints.*

These practical observations combined with certain laboratory experiments intelligently conducted have demonstrated that sulphuric acid is the principal agent in causing decomposition.

CHEMICAL PROCESSES OF DECOMPOSITION

Messrs. Candlot,† Michaelis,‡ and Deval§ have discovered successively by different methods that aluminate of lime $Al_2 O_3 3 CaO$, which exists in cements in company with other calcareous salts, such as silicates, possesses the property of combining with sulphate of lime so as to give a double salt $Al_2 O_3 3 CaO, 3 (SO_3 CaO)$ combined with a large quantity of water with great increase in volume. This substance, moreover, has no firm coherence. It is soluble in pure water, but insoluble in lime water, a fact that explains its existence in a solid state in mortars.

On the other hand, even if the cements do not contain free lime when they are anhydrous, their setting under the action of water frees a part of the lime which was combined with the acid elements, principally with silica. If a soluble sulphate other than sulphate of lime is placed in contact with a hydraulic binding material during hardening or after having set, it produces, with the freed lime, sulphate of lime, which in turn combines with the aluminate, giving "sulpho-aluminate," and produces the swelling which causes the disintegration of the mortar. The same reactions would be produced, moreover, without the intervention of free lime as a result of the reaction of the sulphuric acid of the salt dissolved by the water upon a part of the lime of the binding material.

Although the formation of the sulpho-aluminate of lime seems to be the principal cause of the decomposition of cement by sea water and sulphate waters, it may not be the only one: the setting and the hardening of the cement in contact with water result in the separation of compounds rich in lime, in salts less calcareous, and in free lime. According to the nature of the medium and the conditions affecting its preservation, this reaction may be modified or counteracted in such manner that the hardening cannot follow its regular course; likewise, the lime set at liberty may be dissolved little by little in the water which penetrates the mortars, and may disappear by exosmose, giving place to other more or less injurious compounds.

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*Zamboni, Industria, October 15, 1899.

†Ciments et Chaux Hydrauliques, Paris, 1891, p. 257.

‡Der Cement-Bacillus, Berlin, 1892.

§Bulletin de la Société d'Encouragement pour l'Industrie Nationale, 1900, I, p. 49.

These various phenomena are yet far from being satisfactorily explained; nevertheless, it appears that those cements which are richest in lime are the most quickly decomposed.

SEARCH FOR BINDING MATERIALS CAPABLE OF RESISTING THE ACTION OF SEA WATER

For a long time the efforts of experimenters have been directed toward finding a cement of such composition that it cannot be decomposed by sea water. Thinking at first that the destructive action of the water resulted from the substitution of the magnesia which it contained, for the lime of the cement, the idea was conceived of making cement by burning dolomitic limestone which consequently was composed largely of salts of magnesia. But it was found that the magnesia which this contained, since it was burned necessarily at a very high temperature, was slaked with great difficulty, and by its tardy hydration caused the mortar to swell. Cements were also made experimentally of baryta, a laboratory product whose high price does not permit its introduction into regular practice.*

After the discovery of the sulpho-aluminate of lime, the question changed its aspect, and alumina was considered a dangerous element in cement, the proportion of which ought to be reduced as much as possible. At present the specifications adopted by the Administration of Public Works in France limit to 8% the maximum amount of alumina allowed in cement intended for use in sea water, and this limit would be placed much lower were it not for the fact that in many localities it would be very difficult to obtain products containing less alumina. On the other hand, the percentage of alumina cannot be greatly reduced without at the same time rendering more difficult the burning of the cement, in which operation this element acts as a flux. Accordingly, it was suggested that the alumina be replaced by iron oxide. Cements have been made in the laboratory which were absolutely free from alumina and rich in iron, and these resisted sea water very well.† The various hydraulic cements and limes produced by the works of Teil, whose reputation is world-wide, contain not more than 2% of alumina, and some of them usually last much better in sea water than most of the Portland cements which contain between 7% and 8% of alumina. These too, however, become decomposed under certain conditions, but with this peculiarity — that their disintegration is not usually accompanied by any increase of volume.

*Le Chatelier, Annales des Mines, May and June, 1887.

†Le Chatelier, Congrès International des Matériaux de Construction, held at Paris in 1900, Vol. II, Part 2, p. 51.

It has been noted that the cements which are the richest in lime decompose the most quickly in sea water. Based upon this observation, the experiment was also tried of making cements for marine use by burning mixtures less rich in carbonate of lime than the ordinary Portland cements. This diminished the strength of the cement, but the falling off in strength was only of secondary importance. The principal difficulty lay in the process of manufacture. In burning cements of this class there was produced in the kilns a considerable quantity of powder possessing only a comparatively feeble hydraulic power, which obstructed the draught. This difficulty was lessened by mixing ferruginous materials (ore, etc.), or even sulphate of lime,* with the raw materials before burning. Also, the use of rotary furnaces prevents the choking of the draught. As has just been said, cements low in lime do not attain as great strength as the ordinary Portland cements, but they generally resist the decomposing action of sea water better.

When the proportion of limestone is small, the burning can be done only at a very low temperature, and the cement obtained sets very quickly. Some of these low lime cements appear to resist chemical decomposition satisfactorily, while others resist no better than most of the Portland cements, a difference which has not yet been explained. In any case, on account of the rapidity of set, this class of cements cannot readily be used on large work, and, in fact, their use is mainly limited to special cases.

Another means of neutralizing the bad effects of the excess of lime liberated by the setting of Portland cement consists in mixing with the latter, before using, materials capable of combining with this lime so as to produce insoluble compounds. Puzzolans have been found to be the most useful material for this purpose. Laboratory tests, verified by experiments on a larger scale,† have shown that mortars made in this way generally resist sea water better than if they had been made from similar cements without puzzolanic material. Sometimes, too, their strength is increased by this mixture. It is conceivable, however, that the substances which in the Puzzolans appear as acids are less energetic in their action upon the lime of the cement than the sulphuric acids of sea water or of water containing gypsum, and that therefore in the end they will be displaced by the latter with the consequent decomposition of the mortar. This method cannot then be looked upon as giving absolute security against deterioration although it has been proved to be useful.

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*Candlot, paper delivered at the meeting of the French and Belgian members of the International Association of the Materials of Construction, on April 25, 1903.

†Feret, Annales des Ponts et Chaussées, 1901, IV, p. 191.

METHOD OF DETERMINING THE ABILITY OF A BINDING MATERIAL TO RESIST THE CHEMICAL ACTION OF SULPHATE WATERS

One method is to gage the cement to be tested with sufficient water to obtain a plastic paste, spread this paste on glass plates so as to form cakes or pats with thin edges, immerse the pats in sea water, and observe them from time to time. But with this method the amount of deformation in the pats depends to a large extent upon the hardness of the paste at the time of immersion, so that a cement which cracks when immersed before setting may stand a long time without showing any trace or alteration if the pat is not placed in contact with the water until twenty-four hours after gaging. Further, the surface of the pat is quickly covered by a crust more or less thick resulting from the partial carbonization of the freed lime, so that the substitution of magnesia for a part of this lime and the presence of this crust may influence the decomposition of the underlying cement.

Another and more exact method consists in molding a block of cement or of mortar of a sufficient thickness; for example, a briquette such as is used for a tensile test. Allow this to harden in the usual way, say for twenty-eight days, then cut out from the center of this block a small solid disc with sharp edges, and immerse it in sea water or in a sulphate solution (saturated gypsum, sulphate of magnesia, etc.). In order to prevent all new superficial carbonization of the specimen, carbonic acid should not be allowed to come in contact with or be present in this liquid. When decomposition occurs in the cement it is indicated by cracks which appear at the edge of the disc after a lapse of a variable time.

As a third test, sea water under pressure can be made to filter continuously through mortars made with fine sand. The author of the present chapter uses for this test mortars containing from 250 to 450 kilograms (551 to 991 lb.) of cement per cubic meter (35.3 cu. ft.) of sand (corresponding approximately to proportions 1:6 to 1:3 by weight) which he gages to a plastic consistency and molds into cubes 50 square centimeters (7.74 sq. in.) on a face, with a tube of brass penetrating to the center of the block. After a few days the brass tubes are attached with India rubber tubes to a vessel containing sea water under a head of 2 meters (6.52 ft.). The amount of water which flows through each cube in a given time is accurately measured from time to time, the cube being immersed in sea water in a glass receptacle, where the state of preservation of the mortar can be closely observed.

Finally, the following quite rapid method is used in the laboratory at Boulogne. A mixture is made consisting of 100 parts of cement to be

tested and 300 parts marble ground to a fine powder. To this is added gypsum in the form of a very fine powder, varying progressively from 0% to 20% of the weight of the cement. Plastic mortars are then made from each of these mixtures, which are molded into prisms 2 by 2 by 12.5 centimeters (0.8 by 0.8 by 4.9 in.), allowed to harden for seven days in moist air, and then immersed in fresh water after the length of each has been exactly measured. The water is frequently renewed and at stated periods the lengths of the prisms are again measured, at which time their state of preservation is also examined.

The ability of the cement to resist decomposition by sulphates is indicated by the time taken for the prisms to expand abnormally and to develop cracks, and also by the quantity of gypsum which the binding material is able to bear for a given time without deterioration.

As a result of a long series of experiments, especially of those made by the last two methods, the conclusion has been reached that no binding material has as yet been found which will not be decomposed sooner or later when subjected to these tests, so that at present no cement can be looked upon as absolutely safe from the action of sea water.

MECHANICAL PROCESSES OF DISINTEGRATION

It seems possible to divide the phenomena of disintegration into two classes according as the destruction of the mortar is produced by a sort of progressive dissolution of its elements without appreciable change in volume, or as the products of decomposition, collecting in the pores, enlarge them and produce a scaling off and a weakening of the mortar. This second class of phenomena is much the more frequent and serious.

In both cases decomposition may be produced when the mortar is simply immersed, because of the penetration of the water into its pores and its renewal by the double phenomenon of endosmose and exosmose. But when the masonry is subjected to different degrees of pressure upon its opposite faces, as is usually the case, this tends to establish a current of water through it and the replacement of the dissolving elements goes on more actively. However, disintegration may, under these conditions, proceed more slowly if the current of water is strong enough to carry away the solid products of decomposition as they are formed. The writer has cited in a former paper* experiments which plainly show the difference between these two methods of decomposition: if lean mortars, made with the same cement and sands of different granulometric compositions, are kept in absolutely quiet sea water, those which disintegrate most rapidly are the ones

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*Annales des Ponts et Chaussées, 1892, II, pp. 106 to 116.

into whose composition there enters no fine sand, but only medium sand or, and above all, coarse sand. These latter are the mortars that contain the voids of largest size. On the contrary, if a series of similar mortars are subjected to a continuous filtration of sea water, those made from coarse sand remain intact, while decomposition is more and more active for mortars containing more and more fine sand. *In practise this latter is the most frequent case, and, in fact, it has been verified that the destruction of concrete or mortar by sea water has in most cases been due to the use of too fine sands.*

This is a point which cannot be too strongly insisted upon, and experiments show that a rather lean mortar of coarse sand is much preferable to a mortar of fine sand, even when a very large quantity of cement is introduced into the latter. Fine sands ought to be banished relentlessly from sea water construction even when the cost of coarse sand is very high.* When stone is at hand, an excellent sand can be obtained economically by crushing it.

PROPORTIONS FOR MORTARS AND CONCRETES

From the preceding it is evident that the best means of fighting against sea water is to prevent as far as possible its penetration into the mortars and concretes, and accordingly to make those of great density. The authors of this volume have suggested in a preceding chapter (Chapter IX) with what size of sand and what quantity of cement this result can best be attained in mortars: the maximum density is obtained with a mortar containing sand composed of material having about two parts of very coarse grains to one of fine grains, including cement. Usually, natural sands, even the coarsest, contain a proportion of relatively fine sand sufficient to make it useless to add more with the cement. If a sand is used from which the fine grains have been screened, and this is mixed with about one-half of its weight of cement, a mortar is obtained at once very dense and of great strength, but whose use would often be too costly. In such cases the cement can be replaced by a mixture of sand and cement prepared in advance, such as the product known as "sand-cement," for the making of which a few factories have been built in Europe and also in America. It must be borne in mind, however, that this solution, excellent for mortars destined to remain in the air or to come in contact only with fresh water, would be poor to use in sea water, for very fine sand intimately mixed with cement separates its grains and increases the surface of attack, and various experiments have shown that this kind of mortar suffers severely in sea water.

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*See also, Feret, Baumaterialienkunde, 1896, p. 139, and "Le Ciment," 1896, p. 212.

For use in sea water, on the contrary, if a good puzzolanic material can be procured on favorable terms, it is advantageous to grind this with the cement to take the place of the fine sand, so that in the mortar it may play both a mechanical and a chemical role, assuring to it a great density, and at the same time forming, with the lime freed by the setting, compounds which tend to harden the mortar and render it impermeable.

For concretes the law of greatest density is not the same as for mortars, and it has not yet been possible to express a general law. It is necessary to see that the concrete does not contain voids, and above all that the cement is not diluted by an excess of fine sand, which must always be considered as the greatest enemy of masonry in sea water.

In every case the sea water should be prevented from coming in contact with the work for as long a time as possible, so that the setting of the cement may be already considerably advanced. Yet it must not be forgotten that when the mortar contains a puzzolanic material its hardening can be properly effected only in the presence of moisture.

VARIOUS PLASTERS AND COATINGS

Various methods have been tried to prevent sea water from wetting masonry too soon, either by coating the work with materials designed to obstruct the pores, or by covering it with a layer more or less thick and more or less impermeable, consisting usually of a rich mortar, clay, bituminous materials, etc.

This method of protecting the work is generally rather costly and is not applicable to all kinds of construction. Besides, it presents this disadvantage, that if by accident there is any break in the continuity of the covering, the sea water finds a passage towards the heart of the masonry and creeps in from one place to another, so that often the coating offers only an illusory security.

In certain cases, a coating is formed spontaneously by the carbonization of the lime in the parts of the mortar near the free surface, and this action is aided by the development of sea organisms such as sea-weed and shellfish. This cause, together with the differences in the saltness and the temperature of the water, and the course of the ocean currents, is the one which is most often called upon to explain why mortars decompose more quickly in some regions than in others.

CHAPTER XIX

LAYING CONCRETE AND MORTAR IN FREEZING WEATHER

The results of practise and experiment with cements exposed to frost, which are discussed more in detail in the following pages, may be summarized as follows:

(1) Most Natural cements are completely ruined by freezing. (See p. 410.)

(2) The setting and hardening of Portland cement in concrete or mortar is retarded, and the strength at short periods is lowered, by freezing, but the ultimate strength appears to be but slightly, if at all, affected. (See p. 411.)

(3) A thin scale is apt to crack from the surface of concrete walks or walls which have been frozen before the cement in them has hardened. (See p. 410.)

(4) Frost expands Natural cement masonry and settlement results with the thawing. (See p. 410.)

(5) Heating the materials hastens setting and retards the action of frost. (See p. 413.)

(6) Salt lowers the freezing point of water, and in quantities up to 10% of the weight of the water does not appear to affect the ultimate strength of the concrete or mortar. (See p. 414.)

(7) In practise concrete work should be avoided if possible in freezing weather, because of the difficulty and expense of attaining perfect results. (See p. 410.)

EFFECT OF FREEZING

Numerous experimental tests have been made, chiefly in the United States, where the effect of frost is a more serious question than in England, France, or Germany, to determine the effect of freezing temperatures upon hydraulic cements. Although the conclusions of different experimenters are not in perfect accord, it is the generally accepted belief, corroborated by tests under the most practical conditions and by the appearance of concrete and mortar in masonry construction, that the ultimate effect of freezing upon Portland cement concrete and mortar is to produce only surface injury.

In their practise and research the authors have never discovered a case,

either in laboratory work or in practical construction, where Portland cement concrete or mortar laid with proper care has suffered more than surface disintegration from the action of frost. They do not wish to imply, however, that it is always expedient to lay Portland cement masonry in freezing weather, for the expense of laying is increased, and it is much more difficult to satisfactorily mix and place the materials. Mortar for brick and stone masonry freezes in the tubs and in the joints, while in laying concrete the surface freezes unless measures are taken to prevent it, and any dirt or "laitance" which rises to the surface of wet mixtures is hard to remove. It is a well-known fact that a thin crust about $\frac{1}{16}$ inch thick is apt to scale off from granolithic or concrete pavements which have frozen, leaving a rough instead of a troweled wearing surface, and the effect upon concrete walls is often similar. It may be stated as a general rule that concrete work should, if possible, be avoided in freezing weather, although if circumstances warrant the added expense, with proper precaution and careful inspection mass concrete may be laid with Portland cement at almost any temperature.

Most Natural cements, on the contrary, are seriously injured by frost, especially by alternate freezing and thawing, and while occasional cases are on record, especially in heavy stone masonry in which the weighted joints have thawed slowly, where Natural cement mortar has been laid in freezing weather without serious results, numerous examples might be cited where even after several years the concrete or mortar was but slightly better than sand and gravel. Mr. Thompson has observed this result in Natural cement mortar laid during the comparatively warm winter of North Carolina on days when the temperature was considerably above freezing at the time of laying, and also in the cold climate of Maine where the mortar froze as it left the trowel and did not thaw until spring.

The settlement of the masonry when thawing is often a serious characteristic of Natural cements. Stone masonry walls laid in freezing weather in Natural cement mortar may settle as much as $\frac{1}{2}$ inch in the height of a window jamb.

Experiments upon Natural cement mortars have not positively confirmed the judgment reached by nearly all engineers experienced in construction in freezing weather. Occasional tests are recorded in which such mortars, especially when subjected to a uniformly cold temperature and then suddenly thawed, have attained full strength, but these are insufficient to warrant the use of any except Portland cements when frost is likely to occur before the mortar is thoroughly dry.

The prevention of injury from frost in certain cements may be due, at

least in part, to the internal heat produced when setting. In the interior of a large mass, some cements, especially high grade Portlands, attain a high temperature. (See p. 130.)

Freezing Experiments. An extensive series of experiments upon frozen mortars has been conducted by Mr. Thomas F. Richardson, at the Wachusett Dam in Massachusetts. The results of tests extending up to one year showed that although briquettes mixed 1 part cement to 3 parts sand had less strength at the end of seven days than those which had not been frozen, the frozen specimens after longer periods, especially at the end of one year, gave as high and often higher strength than those which were kept at ordinary temperatures. The conclusion was reached, therefore, that Portland cement mortar is not permanently injured by freezing.

Mr. Richardson's experiments were conducted in the middle of the winter of 1902. He gives the following description* of the tests:

> Two bags of Portland cement were thoroughly mixed together and all the briquettes were made from cement from these bags. Masonry work on the Wachusett Dam was in progress during the period, and briquettes were made each week and submitted to the same conditions as the masonry, the molds being filled with mortar and placed out doors in the air, not in water, immediately after filling.
>
> Briquettes were made at the same time as the ones exposed to the weather, and kept in the laboratory, either in the air or in water, those in the air approximating more closely the conditions which obtained on the masonry construction at the dam. About ½ of the briquettes out doors were exposed to temperatures as low as 9° above zero in the first 24 hours, and some of them to temperatures as low as 12° below zero in the first week. Salt was used in most of the experiments, the quantity ranging from 4 to 16 pounds per barrel of cement, the average being about 6 pounds or about 3% by weight of water. Our experiments indicate that 8 pounds of salt per barrel of cement is sufficient, even in the coldest weather, and the results from 4 pounds are very nearly as good; 16 pounds do not seem to give quite as good results.

The following table gives the average results of the experiments:

Effect of Frost upon Tensile Strength of 1:3 Mortar. (See p. 411.)
BY THOMAS F. RICHARDSON.

Briquettes Kept	No. of Briquettes	Tensile Strength, lb. per sq. in.				
		7 d.	28 d.	3 mo.	6 mo.	1 yr.
Water in laboratory	20	268	304	359	370	401
Air in laboratory	20	298	352	364	392	517
Out doors, below freezing	80	139	238	344	435	627

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*Kindly furnished by Mr. Richardson for this Treatise.

The briquettes were made in sets of 5, consequently 4 experiments are shown for water and air in laboratory, and 16 for out doors.

In France similar results have been reached by Mr. P. Alexandre* as to the effect of temperatures slightly above freezing.

Mr. Charles S. Gowen† also has concluded from his tests that "there is no indication that freezing reduces the ultimate strength of the mortar, although it delays the action of setting."

The effect of different uniform temperatures upon neat cements and mortars is illustrated in Fig. 124, which is selected and adapted by the authors from a series of experiments by Mr. J. E. Howard‡ at the Watertown Arsenal. The results with both neat cements and mortars show but slight increase in strength while the specimens are maintained at 0° Fahr. (—18° Cent.), but a decided increase in strength as soon as they are subjected to a higher temperature. The zero cubes were removed from the freezer and allowed to set one day at 70° Fahr. (21° Cent.) before breaking.

FIG. 124.—Strength of Neat Portland Cement Mortar, 2-inch Cubes, Set in Air at Different Temperatures. (*See p.* 412.)

Cold retards setting. Prof. Tetmajer§ found, for example, that 1:3 Portland cement mortar which attains its initial set at 2¾ hours and its final set at 8½ hours when mixed at 65° Fahr. (18° Cent.), at a temperature of freezing reaches its initial and final set at 21 and 38 hours respectively.

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*Annales des Ponts et Chaussées, 1890, II, pp. 302 and 422.

†Proceedings American Society for Testing Materials, 1903, p. 393.

‡Tests of Metals, U. S. A., 1901, p. 530.

§Johnson's Materials of Construction, 1903, p. 636.

METHODS OF CONSTRUCTION IN FREEZING WEATHER

Certain classes of concrete construction, such as foundations or heavy walls, whose face appearance is of no consequence and which will have opportunity to thaw and then thoroughly harden before loading, may be laid in freezing weather with first-class Portland cement, but it is absolutely necessary to thoroughly remove all dirt and frozen "laitance" (see p. 393) before placing fresh concrete. This is a much more difficult matter than would appear, because frozen dirt has the same appearance as set concrete.

In the case of structures which must not be permitted to freeze, work may often be conducted by maintaining the atmosphere artificially above the freezing point. In temperatures only a few degrees below freezing, it is a common practise to heat the materials, the heat tending both to accelerate the setting of the cement and to lengthen the time before the mixture becomes cold enough to freeze. The addition of salt lowers the freezing point of the water, and therefore of the concrete or mortar.

Protection from Frost. The method of maintaining masonry above the freezing point depends upon the character of the structure. At Beverly, Mass., before beginning the construction of a three-story factory building of concrete, a house of canvas on a light wooden frame was built over the site and braced against wind pressure, so that the concrete was mixed and laid under cover while the temperature was maintained above the freezing point by means of stoves.

A dam was constructed at Chaudiere Falls, P. Q.* when the temperature was 20° below zero. A house 100 feet long by 24 feet wide was built over a portion of the dam in sections about 10 feet square, bolted together, and heated by sheet-iron stoves about 18 inches in diameter by 24 inches high, burning coke. The concrete was mixed and laid in this house, which, when one portion of the dam was completed, was taken down and erected in another place.

A thick covering of straw, sand, or manure may sometimes be effective in preventing freezing. Simply covering with canvas avails but little.

Heating the Materials. Where hand-mixing is employed, an arrangement used on the Newton, Mass., sewers is useful. Sand for one or more batches is placed in a bottomless box containing a coil of steam pipe, the exhaust end of which is then extended to the mixing platform and arranged to discharge through the bottom of the platform into the bottomless box employed for measuring the stone, so that the latter is heated by the exhaust steam. The cement is warmed by piling the bags on top of the sand box.

**Engineering News*, May 7, 1903, p. 402.

An ordinary sand heater, such as is used for asphalt materials, may also be employed, and the stone heated by steam from a hose. A modification of the sand heater,* arranged to form the combined water, sand, and stone heater illustrated in Fig. 125, has been used on the New York Central Railroad.

Experiments by Mr. Thomas F. Richardson† tend to show that heating the materials of mortar has but little, if any, permanent effect upon its strength.

Addition of Salt. Because of its cheapness salt is most commonly employed to lower the freezing point of water. Other materials, such as glycerine, alcohol, and sugar, have been experimentally employed, but these appear to have a tendency to lower the strength of the mortar.

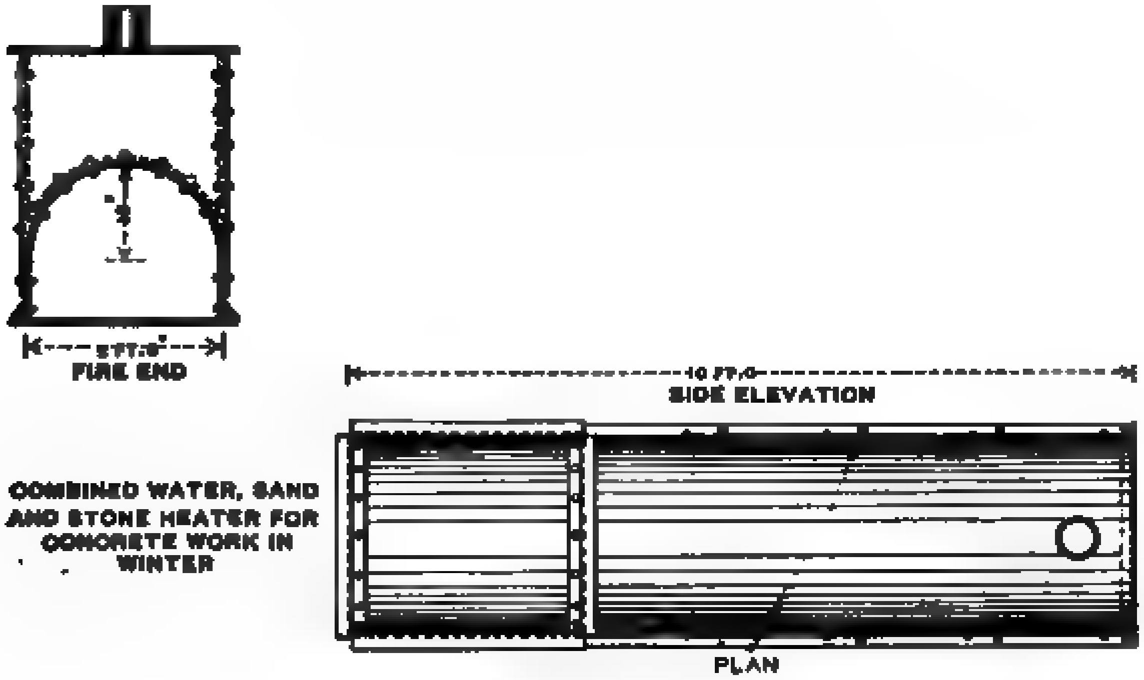

FIG. 125.—Combined Water, Sand, and Stone Heater for Concrete Work in Winter. (*See p.* 414.)

Salt has been more extensively employed in mortars than in concretes. Rules have been formulated for varying the percentage of salt with the temperature of the atmosphere. Prof. Tetmajer's‡ rule, for example, reduced to Fahrenheit units, requires 1% by weight of salt to the weight of the water for each degree Fahrenheit below freezing.

A rule frequently cited in print, which practical tests by the authors have proved to be entirely inadequate, is to require one pound of salt to 18 gallons of water for a temperature of 32° Fahr. and an increase of one

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*George W. Lee in *Engineering News*, March 19, 1903, p. 246.

†Report Metropolitan Water and Sewerage Board, 1904, p. 110.

‡Johnson's Materials of Construction, 1903, p. 615.

ounce for each degree of lower temperature. For 16° Fahr. this corresponds to but slightly more than 1% of the weight of the water, an amount too small to be effective. Since the temperature of the air usually cannot be determined in advance, an arbitrary quantity is as suitable as a variable one. In the New York Subway work in 1903, 9% of salt to the weight of the water was adopted. On the Wachusett Dam, during the winter of 1902, 4 pounds of salt were used to each barrel of cement. For 1:3 mortar this corresponded to about 2% of the weight of the water.

Experiments show that ordinary "quaking" concrete in proportions 1: 2½: 5 requires about 130 pounds of water per barrel of Portland cement, hence 10% of salt in average concrete is equivalent to 13 pounds per barrel of Portland cement. Ordinary 1: 2½ mortar requires about 120 pounds of water per barrel of Portland cement, hence 10% of salt in average mortar is equivalent to about 12 pounds salt per barrel of Portland cement. Salt is sometimes added in sufficient quantity to "float a potato" or an egg. According to tests of the authors, about 15% of salt to the weight of the water is required to float a potato, and about 11% to float an egg.

Recent experiments, by Mr. Gowen* and Mr. Richardson,† extending up to a period of one year, tend to show that salt in a quantity corresponding to at least 10% of the weight of the water does not lower the ultimate strength of ordinary mortar. The time of setting, however, is considerably increased and the strength at short periods is lowered. The effect, at laboratory temperature, of 10% salt with 1: 3 Portland cement mortar is illustrated in the following table:

Tensile Strength of 1:3 Mortars made with Fresh and Salted Water.

By Charles S. Gowen.

	1 week.	1 mo.	3 mos.	6 mos.	9 mos.	12 mos.
Fresh water used..........	112	183	268	335	351	458
Salted water used..........	68	131	215	266	301	413

In Mr. Richardson's experiments‡ smaller percentages of salt proved beneficial. Portland cement mortar in proportions 1: 3, mixed with 4 and 8 pounds of salt per barrel cement (corresponding respectively to about 2% and 4% of the weight of the water), gave slightly higher tensile strength than the unsalted mortar at all periods from 7 days to one year.

Experiments by Mr. E. S. Wheeler§ indicate that the use of 10% of salt tends to prevent the swelling of briquettes in the molds, even if the specimens freeze.

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*Proceedings American Society for Testing Materials, 1903, p. 393.

†Report Metropolitan Water and Sewerage Board, 1903, p. 112.

‡See page 411.

§Report Chief of Engineers, U. S. A., 1895, pp. 2963 to 2971.

CHAPTER XX

WATER-TIGHTNESS

A wall of concrete may be rendered water-tight in several ways:

(1) By accurately grading and proportioning the aggregates and the cement. (See p. 417.)

(2) By special treatment of the surface of the concrete. (See p. 419.)

(3) By the introduction of foreign ingredients into the mixture. (See p. 420.)

(4) By the application of layers of waterproof material, such as asphalt and felt. (See p. 421.)

It is often advisable to combine two or more of these methods.

In the succeeding pages directions are given for practically applying these methods, and experimental investigation is cited.

LAYING CONCRETE FOR WATER-TIGHT WORK

The manner of laying the concrete in walls or floors which are to withstand water pressure is as important as the proportioning of its ingredients. Approved methods of placing are fully described in Chapter XVII.

The chief points applicable to water-tight work are briefly recapitulated as follows:

(*a*) Mix concrete of quaking or of wet consistency. (See p. 416.)

(*b*) Place concrete carefully so as to leave no visible stone pockets.

(*c*) Lay the entire structure, if possible, in one continuous operation, working night and day when necessary.

(*d*) If joints are unavoidable, clean and roughen the old surface, then wet it and coat with a layer of cement or mortar. (See p. 376.)

(*e*) Make suitable provision for contraction by special joints, or by steel reinforcement without joints. (See p. 376.)

Effect of Consistency. A series of experiments, conducted by the authors, upon several blocks of mortar mixed in the same proportions of cement, sand, and stone, but with different proportions of water, indicates that the best consistency for concrete designed to withstand water pressure is intermediate between a *quaking* and a *mushy* mixture, as defined on page 372.

Also, the general conclusion was reached that with the same dry materials the consistency producing the greatest density after setting gives the most

impermeable mortar or concrete up to the point of a very wet consistency, when the excess of water affects the chemical composition of the cement, forming "laitance" (see p. 393), and thus reduces both the strength and the water-tightness of the specimen. After setting, the very wet specimens were found to have about the same density as the medium and mushy mixtures, because the cement, sand, and stone settled into place and expelled the surplus water.

PROPORTIONING WATER-TIGHT CONCRETE

The proportions* employed to resist the percolation of water usually range from 1: 1: 2 to 1: 2½: 4½, the most common mixtures being 1: 2: 4 or 1: 2½: 4½. However, with accurate grading by scientific methods, such as are described in Chapter XI, water-tight work may be obtained with proportions as lean as 1: 3: 7. (See p. 183.) Permeability, the quality of allowing water to pass through, and porosity, the property of containing pores or voids, are not synonymous terms, and the most porous material is not necessarily the most permeable, because the dimensions of the voids as well as their volume affect by capillarity the passage of water.

For maximum water-tightness a mortar or concrete may require a slightly larger proportion of fine grains in the sand than for maximum density or strength, but otherwise the general principles discussed on page 172 are applicable. A mixed aggregate (such as is shown in Fig. 61, p. 173) evidently has fewer channels through which the water can pass than an aggregate consisting of coarse stone and sand (such as is shown in Fig. 59, p. 172), provided the character and relative proportioning of the finest particles are the same in both cases. To carry the comparison still further, a concrete should be less permeable, that is, more water-tight, than its mortar would be if the stone were left out.

Porosity of Concrete. The total voids, air plus water, in first-class concrete and mortar of various proportions are shown in column (20) of the table of Mr. William B. Fuller's experiments on pages 258 and 259. The percentage of total voids in the mortars averages about 26%, while in the concretes, of proportions commonly employed in practice, the voids range from 13% to 17%.

In neither the concrete nor the mortar do these percentages ever represent air alone. A portion of the water, an amount estimated at 8% of the weight of the cement,† corresponding to about 2½% of the volume of

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*Proportions are based on an assumed unit of 100 lb. cement per cu. ft. or the equivalent of 3.8 cu. ft. to the barrel. (See p. 217.)

†Allen Hazen in Transactions American Society of Civil Engineers, Vol. XLII, p. 128.

ordinary concrete, combines with the cement, and a still larger portion of the water remains in the pores unless dried by artificial heat.

The porosity of mortars is discussed on page 127.

Size of Stone. Authorities disagree as to the relative advantages of small stone ranging between ½ and one inch, and coarse stone, ranging from ½ inch up to, say, 2½ inches. The latter is theoretically the better, but it is sometimes claimed that the fine material can be placed more satisfactorily. This depends upon the workmanship. With proper selection of materials and care in laying, the concrete containing the coarse stone produces excellent work, as is illustrated by the constructions at Little Falls, N. J. (see p. 522), and Boonton, N. J. (see p. 496), where carefully graded stone up to 2½ or 3-inch diameter was used.

If very fine stone, under ½-inch, and containing dust, is used for the coarser aggregate, the addition of sand may increase the porosity and the permeability, because concrete with such small stone is practically a mortar, and the finer particles of stone are really sand. A concrete in proportions 1 part cement : 2 parts sand : 4 parts unscreened stone less than ½-inch diameter, makes a porous concrete, while a mixture 1 part cement : 2 parts sand : 4 parts stone ½-inch to 1½-inch diameter, makes a dense one. With the small stone, proportions 1 : 1 : 2 would be the leanest advisable mixture.

The method of proportioning by mechanical analysis, as described by Mr. Fuller in Chapter XI, has been found in practice to produce impermeable concrete.

THICKNESS OF CONCRETE FOR WATER-TIGHT WORK

It is impossible to specify definite thicknesses of concrete to prevent percolation under different heads of water, because of variations in proportions and methods of laying. We have known rain water under a head of 2 or 3 inches to percolate through a 4-foot wall of excellent concrete of dry consistency. On the other hand, had the same materials been mixed to a wetter consistency and placed with no joints between successive layers, concrete but a few inches thick would have withstood a high head.

The best criterions for thicknesses of walls of first-class concrete are obtained from actual examples. Instances are cited on pages 497, 522, and 523 of water-tight concrete 4 inches thick sustaining a head of 4 feet, concrete 15 inches thick sustaining a head of 40 feet, and concrete 5.5 feet thick sustaining a head of 100 feet.

SPECIAL TREATMENT OF SURFACE

Various methods of treating the surface of concrete have been employed to increase the water-tightness.

Plastering. Plastering the surface of concrete with rich Portland cement mortar in proportions 1 : 1 or 1 : 1½ is the method which first occurs to one, but in temperate or cold climates it is only useful for walls below the surface of the ground and therefore not subject to atmospheric changes. In such cases it can sometimes be used as a substitute for, or in connection with, paper and asphalt.

In certain sections of the Boston Subway, a 6-inch wall of concrete was laid up next to the bank of earth and plastered with a layer of 1 : 1 mortar about ½ inch thick. After spreading the mortar with a plasterer's ordinary metal float (see Fig. 126, p. 443) the surface was run over with a toothed roller about 12 inches long by 4 inches in diameter, which pressed the plaster into any crevices, and left a rough surface. The main wall of concrete forming the lining of the Subway was then laid up against this plastered surface.

On the arch of the approaches to the East Boston tunnel, a layer of plaster, like that on the walls, was spread before laying the final 6-inch thickness of concrete, thus forming a water-tight joint in the interior of the arch ring.

Granolithic Finish. On horizontal or inclined surfaces, a granolithic surface of rich mortar of Portland cement and sand, or Portland cement and screenings in proportions about 1 : 1 may be laid and troweled, as in sidewalk construction. (See Chapter XXII.) The surface finish must be placed at the same time as the base, and with the same, that is, Portland cement.

Troweling Surface. The water-tightness of horizontal or inclined layers of concrete can be greatly increased by troweling the concrete in the same manner that granolithic work is troweled. (See page 443.) This brings the cement to the surface, and produces a dense, hard surface which is nearly equal to a surfacing of rich mortar. This is very effective for surfacing a structure like the inclined face of the dam shown in Fig. 156, page 497.

In experimenting upon the permeability of different concretes, the authors have noticed that even the very light joggling which is necessary to compact a wet concrete, and also the ramming of a stiffer mixture, increases the impermeability of the concrete. Even after chipping off the top of the specimen for a depth of ⅛ or ¼ of an inch, the flow will be several times less than when the pressure is directed upon its under surface.

Grout. Portland cement grout is preferable to plaster for coating the soffits of arches or for wall surfaces. It is also valuable for coating the interior of cisterns or tanks.* The grout should of course be applied against the surface which is to come in contact with the water, and if the wall is to be made impervious in both directions, both sides should be washed.

Bridge specifications of Mr. Edwin Thacher† require: that the top surfaces of the arches, piers, and abutments, and the lower 6 inches of the inner surface of the spandrel walls, shall be coated with a heavy coat of semi-liquid mortar consisting of one part cement, one-half part thoroughly slaked lime, and three parts sand, spread to leave a smooth finish; and after this has set hard it shall be given a heavy coat of pure cement grout.

A specially prepared cement wash has been found effective in preventing dampness in masonry.‡

INTRODUCTION OF FOREIGN INGREDIENTS

The principal advantage of introducing foreign ingredients into a mortar or concrete is to permit the use of a lean mixture, the fine particles of hydrated lime, or whatever may be used, tending to reduce the volume and the dimensions of the voids.

Lime and Puzzolan Cement. The effect of the addition of lime in small quantities is chiefly mechanical, and the quantity which should be employed depends, therefore, upon the fineness of the sand and the proportions of the mixture.

It appears impossible to replace the water which separates the grains of cement or cement and sand in neat paste or 1 : 1 or 1 : 2 mortar with a material like lime. It can only be used to advantage, therefore, with mortars leaner than 1 : 2. In the authors' tests 1 : 2½ : 5 concrete was made more water-tight, although its strength was slightly reduced, by substituting an equal weight of lime paste for 10% by weight of the cement.

The effect of the addition of lime upon the strength and density of mortar is discussed on page 154.

Unslaked lime must not be used under any circumstances. (See p. 156.)

Puzzolan cement, unlike lime, tends to increase the strength even of neat cement and rich mortars,§ in many cases 20% by weight of total dry materials being beneficial if the Puzzolan cement is ground with the Portland.

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*J. W. Schaub, Transactions American Society of Civil Engineers, Vol. LI, p. 123.

†Cement, May, 1903, p. 107.

‡Oscar Lowinson, Transactions American Society of Civil Engineers, Vol. LI, p. 125.

§Feret's Chimie Appliquée, 1897, p. 493.

Undoubtedly the impermeability is similarly increased, since mixtures of Portland and Puzzolan cements have been found to well resist the action of sea water.*

Pulverized Rock. Mortars 1:3 and leaner, and concrete made with these proportions of cement and sand to the stone, are increased in strength,† and probably in impermeability, by the addition of rock pulverized as finely as the cement and equal to it in weight, although if the natural sand is very fine or contains dust, the addition of fine material is not beneficial.

Alum and Soap. A soap and alum mixture in various proportions sometimes is used to make what is called "waterproof mortar." The Sylvester Process mixture employed in New York Harbor by Major W. L. Marshall‡ was made by "taking one part cement and 2½ parts sand and adding thereto ¾ of a pound of pulverized alum (dry) to each cubic foot of sand, all of which was first mixed dry, then the proper amount of water—in which had been dissolved about ¾ of a pound of soft soap to the gallon of water—was added, and the mixing thoroughly completed. The mixture is little inferior in strength to ordinary mortar of the same proportions and is impervious to water, and is also useful in preventing efflorescence."

The effect of alum and soap in diminishing the permeability has been experimented upon by Mr. Edward Cunningham§ and Prof. W. K. Hatt,§ and found useful for small structures.

LAYERS OF WATERPROOF MATERIAL

The use of cement plaster has already been described on page 419.

Layers of waterproof paper or felt cemented together with asphalt or bitumen or tar are extensively used,—and sometimes asphalt alone,—to form an impervious layer. A mixture of alum and lye has also been tried.

Paper or Felt Waterproofing. Layers of paper or felt with tar or asphalt between them are employed for a waterproof course in concrete floors, roofs, and walls of underground structures of large or long area, like tunnels and subways, which require special protection from infiltration of water. The materials range from ordinary tarred paper, laid with coal tar pitch, to asbestos or asphalted felt, laid in asphalt. Coal tar products will deteriorate when continually exposed to moisture, and are therefore not adapted for important locations.

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*See R. Feret, Chapter X, also in Annales des Ponts et Chaussées, 1901, IV, p. 194.

†Feret's Chimie Appliquée, 1897, p. 477.

‡Report Chief of Engineers, U. S. A., 1901, p. 918.

§Transactions American Society of Civil Engineers, Vol. LI, pp. 127 and 128.

In the New York Subway, portions of which are built below tide-water, much of the waterproofing consists of layers of felt laid in asphalt. The specifications,* approved by Mr. William Barclay Parsons, Chief Engineer, contain the following requirements for the materials:

> The asphalt used shall be the best grade of Bermudez, Alcatraz, or lake asphalt, of equal quality, and shall comply with the following requirements: The asphalt shall be a natural asphalt or a mixture of natural asphalts, containing in its refined state not less than ninety-five (95) per cent. of natural bitumen soluble in rectified carbon bisulphide or in chloroform. The remaining ingredients shall be such as not to exert an injurious effect on the work. Not less than two-thirds (⅔) of the total bitumen shall be soluble in petroleum naphtha of seventy (70) degrees Baumé or in Acetone. The asphalt shall not lose more than four (4) per cent. of its weight when maintained for ten (10) hours at a temperature of three hundred (300) degrees Fahrenheit.
>
> The use of coal tar, so-called artificial asphalts, or other products susceptible to injury from the action of water, will not be permitted on any portion of the work, or in any mixtures to be used.
>
> The felt used for waterproofing shall be dipped in asphalt and weigh not less than fifteen (15) pounds to the square of one hundred (100) feet. All felt shall be subject to the inspection and approval of the Engineer.

Method of Laying Paper or Felt. The waterproof layer of a floor may be laid directly upon the ground if the soil is fairly dry and firm, but is usually spread upon a layer of concrete from 4 to 8 inches thick. In the former case† the first layer consists of strips with a 2 to 6-inch lap cemented with asphalt, and the remaining layers are mopped on. Upon a concrete base it is customary to first spread a layer of asphalt upon the concrete, although, if the concrete is damp, the bottom layer of paper or felt may be placed dry, as described above.

The "ply" in waterproofing, — that is, the number of layers which cover all parts of the surface, — varies from 2-ply to 10-ply. It is considered better practice to "shingle" the strips than to place each ply or layer independently. If the surface to be waterproofed is rough it may be leveled with cement mortar. It must be dry before applying the tar or asphalt. The asphalt is heated and brought, generally in buckets, to the work. Several rolls of paper are started consecutively. Ahead of each roll, as it is unrolled, the liquid asphalt is swabbed upon the concrete with a mop, so that the paper or felt is spread directly upon the fresh hot stuff. As soon as the first roll is started the second is placed to overlap the first,

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*Contract No. 2, June, 1902, p. 107.

†This method was followed in portions of the floor in the approaches to the East Boston Tunnel.

a width depending upon the number of ply to be laid. For example, if the felt is 32 inches wide and is laid 3-ply, the second roll is lapped upon the first about 22 inches. As this is unrolled (in the same general direction as the first roll) the surface ahead of it is mopped with asphalt, as described above. A third roll is immediately started, lapping both of the two others, and so on for the entire width of the surface to be covered.

A waterproof course of this character always forms a distinct joint in the mass, thus destroying its cohesion upon that plane, and the strength of the concrete in bending on the two sides of the layer must be considered independently.

New York Subway Specifications.* The specifications for materials are quoted on the preceding page. With reference to the laying the contract requires:

> Each layer of asphalt fluxed as directed by the Engineer must completely and entirely cover the surface on which it is spread without cracks or blowholes.
>
> The felt must be rolled out into the asphalt while the latter is still hot, and pressed against it so as to insure its being completely stuck to the asphalt over its entire surface, great care being taken that all joints in the felt are well broken, and that the ends of the rolls of the bottom layer are carried up on the inside of the layers on the sides, and those of the roof down on the outside of the layers on the sides so as to secure a full lap of at least one (1) foot. Especial care must be taken with this detail.
>
> None but competent men, especially skilled in work of this kind, shall be employed to lay asphalt and felt.
>
> When the finishing layer of concrete is laid over or next to the waterproofing material, care must be taken not to break, tear, or injure in any way the outer surface of the asphalt.
>
> The number of layers of felt on the sides and under the floor shall in no case be less than three (3) in ground that is quite dry, and where there is a water pressure against the masonry equal to ten (10) feet not less than six (6) layers. Where the water pressure is less than ten (10) feet, such number of layers between three (3) and six (6) shall be used as the Engineer may direct. The number of layers of felt on the roof shall be not less than four (4).
>
> Whenever the pressure of ground water against the structure exceeds ten (10) feet, waterproofing of the floor and walls shall then consist of two (2) layers of felt in asphalt, as described above, together with one (1) or more layers of brick dipped in asphalt as ordered by the Engineer. Said bricks before being dipped in asphalt shall be thoroughly dried and warmed. At all other points where the pressure of ground water is less than ten (10) feet, the Contractor may substitute in lieu of the number of layers of felt,

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*Contract No. 2, June, 1902, p. 109.

as described above, one (1) layer of felt in hot asphalt, and one (1) or more courses of brick dipped in asphalt, as the Engineer shall direct.

In masonry-lined structures where there is no steel work and the ground is dry the regular waterproofing may be omitted, but in that case in arched cut and cover work the extrados of the arch shall be coated with hot asphalt of the quality described.

Any masonry that is found to leak at any time prior to the completion of this work shall be cut out and the leak stopped.

Asphalt Waterproofing. Asphalt is sometimes laid as a waterproof course in one or more continuous sheets, and is also used for filling contraction joints in concrete.

In the sedimentation basin for the Albany (N. Y.) Filtration Plant[*] 16 inches of clay and gravel puddle were covered with 6 inches of concrete laid in blocks 7 feet square, with ½-inch asphalt joints 3 inches deep, that is, extending half-way through the concrete. This proved to be a successful treatment.

In the Astoria (Ore.) Water Works[†] the bottom of the reservoir consisted of 6 inches of concrete in approximate proportions, one packed cement : 0.7 sand : 3.5 fine gravel : 6.5 broken stone, covered with a ⅜-inch finishing coat of 1 : 1 mortar and upon this two layers of Alcatraz brand asphalt. The first layer was of natural liquid asphalt, and the second was the product of refining natural rock asphalt with about 20% of the liquid as a flux. Mr. Adams made the rule that no asphalt should be placed until after the concrete had set at least two weeks, and was well dried out. All dust was carefully removed from the concrete, and the asphalt was applied with twine mops. The slopes of the reservoir were lined with brick laid in asphalt upon 6 inches of concrete. Under ordinary conditions such complete measures are unnecessary.

In the construction of government fortifications by the United States Army Engineers, numerous methods of waterproofing have been used,[‡] in some cases an asphalt course being placed between two layers of concrete. Asphalt paint has been used for a protective coating where earth is to be deposited above or against it.[§]

A ¼-inch coating of asphalt applied hot with a mop upon a surface already covered with grout (see p. 420) has been satisfactorily used by

[*] Allen Hazen in Transactions American Society of Civil Engineers, Vol. XLIII, p. 258.

[†] Arthur L Adams in Transactions American Society of Civil Engineers, Vol. XXXVI, p 29.

[‡] Report Chief of Engineers, U. S. A., 1901, pp. 911 to 925, and 1902, pp. 2451 to 2484.

[§] Report Chief of Engineers, U. S. A., 1902, p. 2473.

Mr. J. W. Schaub* for coating the interior of tanks where the head is greater than 10 feet. He considers this sufficient to withstand a water pressure of 60 feet.

Mr. Schaub* also suggests the method of building the wall in two parts and filling the core or hollow space between with asphalt.

Alum and Lye Waterproof Wash. The United States Army Engineers† have sometimes satisfactorily employed for a waterproof wash a mixture of concentrated lye and alum in proportions one pound lye to five pounds alum with proper precautions in mixing and placing it.

RESULTS OF EXPERIMENTS ON PERMEABILITY

Mr. R. Feret‡ in an extended series of experiments upon cement mortars reached the conclusions (*a*) that with mortars of the same granulometric composition (see p. 141) the most impermeable were those which contained the largest percentage of cement, (*b*) of mortars containing the same percentage of cement, but of variable granulometric composition, the most impermeable were those containing equal parts of coarse grains, G, and fine grains, F (see p. 142), the latter including the cement, (*c*) decomposition by the passage of sea water through mortars mixed in equal proportions by weight increases as the sand contains more fine grains.

Mr. Paul Alexandre§ found that mixed sands made a much more impermeable mortar than fine or coarse alone.¶ Mr. Thomas F. Richardson reached similar conclusions, and contrary to the French experiments, his results indicate in general that mixtures giving maximum strength also give maximum impermeability.

The consistency in mixing the concrete affects the permeability only indirectly. Experiments by the authors indicate in general that the mixture giving the greatest density is apt to be most water-tight.

METHODS OF TESTING PERMEABILITY

The relative permeability of different specimens of concrete may be tested by a method similar to that employed by the French Commission for mortar, although the area of surface in contact with the material (the area of the tube) is so small that impurities in the water are liable to fill the

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*Transactions American Society of Civil Engineers, Vol. LI, p. 123.

†G. B. Hegardt in Report Chief of Engineers, U. S. A., 1902, p. 2482.

‡Annales des Ponts et Chaussées, 1892, II, p. 109.

§Annales des Ponts et Chaussées, 1890, II, p. 407.

¶A portion of Mr. Alexandre's experiments are tabulated by Sanford E. Thompson in Transactions American Society of Civil Engineers, Vol. LI, p. 132.

pores and affect the results of time tests. For rough comparative tests the authors have satisfactorily employed discs about 9 inches in diameter and 8 inches thick, to the surface of which a 1-inch iron pipe is cemented in a similar manner to that shown in Fig. 47, page 128. An ordinary pipe flange is screwed on to the pipe, so that the end of the pipe projects through it about ¼ inch. A ¼-inch layer of neat cement in a stiff paste is plastered upon the under side of the flange (using care not to close the end of the pipe), and the pipe is set upon the specimen and a cone of neat cement formed upon the top of the flange, and around it and the pipe so as to cover the entire surface of the specimen. After setting for a few days the specimen can be quite roughly handled without breaking the joint between the neat cement and the concrete or the cement and the pipe.

The method of immersing the specimen in water advocated by the French Commission was found unsatisfactory for tests of short duration. The specimen was therefore soaked in water for twenty-four hours, removed from the water just before the test was commenced and allowed to drain one minute, when the pipe was connected with the pressure, about 80 pounds. The specimen was suspended over a receptacle, and the water passing through it was weighed at intervals. In some cases an idea of the amount of water passing was quickly obtained by counting the drops per minute or noting by a stop-watch the time between the drops.

By molding the specimen of concrete or mortar in an iron pipe* or in a casing of neat cement,† and connecting the end with the water pressure, a larger surface of water contact may be given to the specimen, and the thickness of the material through which the water passes may be more definitely defined. A difficulty encountered when employing an iron pipe is the shrinkage of the cement on setting which is liable to separate the specimen from the tube. On long time tests, loss by evaporation must be prevented.

*Method adopted by Mr. Thomas F. Richardson.

†Method adopted by Mr. William B. Fuller.

CHAPTER XXI

FIRE AND RUST PROTECTION

Observations of steel imbedded in concrete which has been exposed to fire or to corrosive action, and experimental tests prove conclusively that 1½ to 2 inches of dense Portland cement concrete, made in ordinary proportions, with broken stone, gravel, or cinders, of good quality, and mixed wet, will effectually resist the most severe fire liable to occur in buildings, and will prevent the corrosion of steel even under extraordinary conditions. In members of inferior importance or which are only liable to fire of comparatively low temperature, a less thickness of concrete, in many cases ¾-inch or even ½-inch, will prove effective. (See p. 433.)

In buildings concrete has been found a more effective fire-resisting material than terra-cotta (see p. 433) and fully equal to first-class brickwork. Brickwork cannot exist in a structure except in combination with some other material like steel or wood, which is seriously affected by fire, whereas concrete reinforced with steel may replace not only the brickwork, but also the steel or wood columns and beams.

PROTECTION OF STEEL BY CONCRETE

Tests by Prof. Charles L. Norton

Extended practical tests have been conducted by Prof. Charles L. Norton for the Insurance Engineering Station in Boston. As a result of experiments made in 1902 upon several hundred specimens, he concludes:*

(1) Neat Portland cement, even in thin layers, is an effective preventive of rusting.

(2) Concretes, to be effective in preventing rust, must be dense and without voids or cracks. They should be mixed quite wet where applied to the metal.

(3) The corrosion found in cinder concrete is mainly due to the iron oxide, or rust, in the cinders, and not to the sulphur.

(4) Cinder concrete, if free from voids and well rammed when wet, is about as effective as stone concrete in protecting steel.

In his first series of experiments, round rods of mild steel, soft sheet steel, and expanded metal were each imbedded in the center of blocks of

**Engineering News*, October, 1902, p. 334.

concrete, 3 by 3 by 8 inches. Neat cement, 1:3 mortar, and concrete in proportions 1 cement : 5 broken stone; 1 cement : 7 cinders; 1 cement : 2 sand : 5 broken stone; and 1 cement : 2 sand : 5 cinders, were employed for imbedding the steel. The stone was chiefly of trap rock. These specimens, after setting, were subjected continuously to the action of steam, air, and carbon dioxide. Unprotected pieces of steel were also exposed to the same test.

At the end of three weeks the unprotected pieces of steel "were found to consist of rather more rust than steel." The protection of the steel incased in neat cement was perfect. The remaining specimens, in mortar and concrete, were seriously corroded in spots, but it was observed that the "rust spot was invariably coincident with either a void in the concrete or a badly rusted cinder. In the more porous mixtures, the steel was spotted with alternate bright and badly rusted areas, each clearly defined." One point is exceedingly instructive:

> In both the solid and the porous cinder concretes, many rust spots were found, ***except where the concrete had been mixed very wet, in which case the watery cement had coated nearly the whole of the steel, like a paint, and protected it.***

Protection of Rusty Steel. In 1903, Prof. Norton made tests to determine the protection afforded ordinary rusty or dirty steel. He found that while unprotected steel "vanished into a streak of rust," if protected by an inch or more of sound concrete, not only the sound steel but ordinary structural steel of any degree of cleanliness likely to be in use in a building is unaffected by such extreme treatment as was accorded it in the tests. The conditions of these later experiments were similar to those of the previous year. Each piece of steel was stamped, and this removed loose scale. Dirt was removed by a soft wire brush. The steel was imbedded to a depth of 1½ inches in all directions in broken stone concrete of proportions 1:2½:5 and in cinder concrete of proportions 1:3:6. The treatment of the specimens was similar to that of the previous ones.

A portion of Prof. Norton's conclusions* are given in the following paragraphs:

> **Condition of Specimens.** After varying lapses of time from one to three months for the specimens in the "corroders," and from one to nine months for the others, the specimens were broken out of the briquettes cleaned by brushing, and weighed and calipered. Not one specimen had

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**Engineering News*, January, 1904, p. 30.

shown any sensible change in weight or dimension, except where the concrete had been poorly applied. Some specimens were purposely bedded in very dry concrete, and some in concrete partly set, and many of these were not well covered and the steel was seriously attacked where there were voids or cracks. Of the hundreds of specimens of rusty steel examined, not one which had a continuous unbroken coating of concrete gained or lost anything in volume or weight by treatment which caused the practical destruction of some of the unprotected specimens. If loss by corrosion as great as 1-1000 of the loss occurring with the unprotected specimens had been experienced in the case of the protected pieces it would have readily been noted.

Conclusions. It would therefore seem that if we admit that from a severe trial of a short duration, we may judge relatively of the effects of the less severe but longer test of time, it can not be questioned that structural steel is safe from corrosion if incased in a sound sheet of good concrete, at least for a period of years so long as to make the subject of more interest to our great-grandchildren's children than to us. We know that bare steel does not rust and fall down over night, and that much of the steel standing has been bare of everything that could protect it, for long years, and it seems to me beyond question that steel properly covered in concrete may well be expected to last far longer than the changes in our cities will allow any building to remain.

Protection by Cinder Concrete. There is one limitation to the whole question, that is the possibility of getting the steel properly incased in concrete. Many engineers will have nothing to do with concrete because of the difficulty in getting "sound" work. This is especially true of cinder concrete, where the porous nature of the cinders has led to much dry concrete and many voids, and much corrosion. I feel that nothing in this whole subject has been more misunderstood than the action of cinder concrete. We usually hear that it contains much sulphur and this causes corrosion. Sulphur might, if present, were it not for the presence of the strongly alkaline cement; but with that present the corrosion of steel by the sulphur of cinders in a sound Portland concrete is the veriest myth, and as a matter of fact the ordinary cinders, classed as steam cinders, contain only a very small amount of sulphur. There can be no question that cinder concrete has rusted great quantities of steel, but not because of its sulphur, but because it was mixed too dry, through the action of the cinders in absorbing moisture, and that it contained, therefore, voids; and secondly, because in addition the cinders often contain oxide of iron which, when not coated over with the cement by thorough wet mixing, causes the rusting of any steel which it touches.

Mix Wet. There is one cure and only one, *mix wet* and mix well.* With this precaution I would trust cinder concrete quite as quickly as stone concrete in the matter of corrosion.

Rust no Protection for Steel. It has been suggested that steel which has been rusted to a slight depth becomes protected by this coating from further rusting. Nothing could be further from the truth. A large num-

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*See page 372 for the authors' definition of a very wet mixture.

ber of specimens were rusted by repeated alternate wetting and drying to see if they finally reached a constant condition. Instead of doing this, they all showed an irregular but persistent loss in weight, on further rusting, until some had practically been washed away.

Small Rods. The increasing use of steel of small dimensions in floors and roofs, twisted rods, expanded metal, etc., has caused some question as to the advisability of their use in view of the possible great effects of corrosion, as compared with the effects of corrosion on larger members, but with sound concrete of a thickness of about 1½ in. between the steel and the weather I do not question the durability of these lighter members.

CHEMICAL UNION OF STEEL AND CEMENT

Experiments of Mr. Breuillé* indicate that clean steel may form with cement a chemical combination which is soluble in water. This presents an additional reason for making concrete in which steel is imbedded as impervious as possible, to avoid the penetration of moisture which will wash away this chemical compound, if such is found to exist in actual structures. Large I-beams imbedded in concrete would be especially subject to deterioration from this cause, but as rust rarely forms between two plates of steel which are riveted together in a bridge, even although the rest of the structure is badly corroded, the danger is probably insignificant.

Cement Paint for Protecting Steel. The property of neat cement which prevents steel from corrosion is taken advantage of in different forms of cement coating. Mr. Maximillian Toch in 1903† made a series of experiments upon metal covered with various preparations of cement, and drew the following conclusions:

(1) A proper cement paint can be applied to a surface that has begun to oxidize, and further oxidation will be arrested.

(2) If the cement be absolutely fine and free from iron, calcium sulphate and sulphites, and of low specific gravity, it will set on the surface within a very short time, and eventually become an integral part of the metal.

For exposed iron work Mr. Toch recommends a protective coat of cement paint followed by a coat of linseed oil paint. To protect from the fumes of a factory, he states that after applying three coats of cement paint, an alkali-proof, adherent paint may be spread, and an absolute protection afforded to the iron.

Mr. J. W. Schaub‡ refers to the use of cement mortar in Europe and in

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*J. W. Schaub in Transactions American Society of Civil Engineers, Vol. LI, p. 124.

†Lecture on the Permanent Protection of Iron and Steel, delivered before the New York Section of the American Chemical Society, March 6, 1903.

‡*Engineering News*, June 16, 1904, p. 561.

the United States for coating iron exposed to destructive agencies. He says:

> The mortar is usually a mixture of 1 cement and 2 sand, applied with a brush as a wash. Five or six coats are applied in this way to give the metal a proper coating. This is especially applicable in the case of the iron work exposed in roundhouses, where the gases from locomotives are so destructive, and where paint is so inefficient.

FIRE PROTECTION

Numerous experimental tests* have been made showing the value of concrete as a fire-resisting material, but the best proof of its ability to resist the heat of a severe fire — such as is liable to occur in an office or factory building — lies in the fact that concrete has actually withstood very severe fires more successfully than have terra-cotta and various other so-called fireproof materials.

The reinforced concrete factory of the Pacific Coast Borax Co. at Bayonne, N. J., passed through a severe fire in 1902. Still more recently, in 1904, occurred the conflagration at Baltimore in which many building materials utterly failed.

Such practical tests, further confirmed by numerous experiments with test buildings of reinforced concrete, have proved that while in a severe fire, where the temperature ranges from 1600° to 2000° Fahr., the surface of the concrete may be injured to a depth of from ½ to ¾ inch, the body of the concrete is unaffected, so that the only repairs required consist of a coating of plaster, and even this only in rare instances.

Tests upon small briquettes of cement placed in a furnace indicate that the strength of cement is destroyed by a heat reaching a dull, red color,† but as stated below, in an actual fire, the injured material protects the rest of the concrete so that the danger is theoretical rather than real.

Fire in Borax Factory. The fire in the 4-story reinforced concrete factory of the Pacific Coast Borax Company,‡ built entirely of concrete except the roof, utterly destroyed the contents of the building, the roof, and the interior framework, but the walls and floors remained intact except in one place where an 18-ton tank fell through the plank roof and cracked some of the floor beams, and in one place on the outside of the wall where the surface of the concrete was slightly affected. The fire was so hot that brass and iron castings were melted to junk. A small annex,

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*See References, Chapter XXIX.

†Digest of Physical Tests, Vol. I, p. 217.

‡See p. 463.

built of steel posts and girders, was completely wrecked, and the metal bent and twisted into a tangled mass.

Baltimore Fire. The effect of the fire upon the concrete in various buildings located in the center of the burned districts of Baltimore is best appreciated by an examination of the reports of experts upon the fire. Capt. John S. Sewell, in his report to the Chief of Engineers, U. S. A.,* in referring to the fire in one of the buildings built with reinforced concrete columns, beams, and arches, writes:

> It was surrounded by non-fireproof buildings, and was subjected to an extremely severe test, probably involving as high temperature as any that existed anywhere. The concrete was made with broken granite as an aggregate. The arches of the roof and the ceiling of the upper story were cracked along the crown, but in my judgment very slight repairs would have restored any strength lost here. Cutting out a small section — say an inch wide — and caulking it full of good strong cement mortar would have sufficed. The exposed corners of columns and girders were cracked and spalled, showing a tendency to round off to a curve of about 3 in. radius. In the upper stories, where the heat was intense, the concrete was calcined to a depth of from ¼ to ¾ inch, but it showed no tendency to spall, except at exposed corners. On wide, flat surfaces, the calcined material was not more than ¼-inch thick, and showed no disposition to come off. In the lower stories, the concrete was absolutely unimpaired, though the contents of the building were all burned out. In my judgment, the entire concrete structure could have been repaired for not over 20% to 25% of its original cost. On March 10, I witnessed a loading test of this structure. One bay of the second floor, with a beam in the center, was loaded with nearly 300 pounds per sq. ft. superimposed, without a sign of distress, and with a deflection not exceeding ⅛-inch. The floor was designed for a total working load of 150 pounds per sq. ft. The sections next to the front and rear walls were cantilevers, and one of these was loaded with 150 pounds per sq. ft. superimposed, without any sign of distress, or undue deflection.

Captain Sewell concludes as a result of the examination of this and other buildings containing reinforced concrete construction:

> As the material is calcined and damaged to some extent by heat, enough surplus material should be provided to permit of a loss of say ¾-inch all over exposed surfaces, if the structure is to be exposed to fire; moreover, all exposed corners should be rounded to a radius of about 3 inches. This latter precaution would add much to the resistance of all materials used in masonry — whether bricks, stone, concrete or terra-cotta — if they are to be exposed to fire.

**Engineering News*, March 24, 1904, p. 276.

Concrete Versus Terra-Cotta. Prof. Norton, in his report on the Baltimore fire to the Insurance Engineering Experiment Station,* says:

> Where concrete floor arches and concrete-steel construction received the full force of the fire it appears to have stood well, distinctly better than the terra-cotta. The reasons I believe are these: First, because the concrete and steel expand at sensibly the same rate, and hence when heated do not subject one another to stress, but terra-cotta usually expands about twice as fast with increase in temperature as steel, and hence the partitions and floor arches soon become too large to be contained by the steel members which under ordinary temperature properly enclose them. Under this condition the partition must buckle and the segmental arches must lift and break the bonds, crushing at the same time the lower surface member of the tiles.
>
> When brick or terra-cotta are heated no chemical action occurs, but when concrete is carried up to about 1 000° Fahr. its surface becomes decomposed, dehydration occurs, and water is driven off. This process takes a relatively great amount of heat. It would take about as much heat to drive the water out of this outer quarter-inch of the concrete partition as it would to raise that quarter-inch to 1 000° Fahr. Now a second action begins. After dehydration the concrete is much improved as a non-conductor, and yet through this layer of non-conducting material must pass all the heat to dehydrate and raise the temperature of the layers below, a process which cannot proceed with great speed.

Cinder Versus Stone Concrete. Prof. Norton compares the action of stone and cinder concrete in the Baltimore fire as follows:

> Little difference in the action of the fire on stone concrete and cinder concrete could be noted, and as I have earlier pointed out, the burning of the bits of coal in poor cinder concrete is often balanced by the splitting of the stones in the stone concrete. I have never been able to see that in the long run either stood fire better or worse than the other. However, owing to its density the stone concrete takes longer to heat through.

Further experiments are required to determine the relative durability under extreme heat of concrete made with different kinds of broken stone. It seems probable, from the composition of the rock, that hard trap or gravel may be preferable to limestone, slate, or conglomerate as fire-resisting material.

Thickness of Concrete Required to Protect Metal from Fire. The conclusion reached by Prof. Norton† from tests upon concrete arches is that two inches of good concrete gives perfect assurance of safety in case of fire, even if the steel to be protected is in the form of I-beams. Rods of

**Engineering News*, June 2, 1904, p. 529.

†*Insurance Engineering*, Dec., 1901, p. 483.

small dimensions can be more effectively coated, and it appears evident from the various tests and from practical experience in severe fires that 1½ inches of concrete around steel rods is sufficient protection. The Pacific Borax Company's fire and other similar tests indicate that in slabs of reinforced concrete, ½ inch to ¾ inch affords ample protection. Secondary members, such as cross girders, or slabs of long span, should have a thickness of concrete outside of the steel varying from ¾ inch to 1½ inch. Although in slabs protected by only ½ inch of concrete, the latter may be softened by an extreme fire, and the metal exposed when it is struck by the stream from a hose, the metal in the majority of cases would still remain practically uninjured, and it is questionable economy to put an excess of material where there is so little probability of its being needed, and where a failure would merely produce local damage.

THEORY OF FIRE PROTECTION

Mr. Spencer B. Newberry, in an address delivered before the Associated Expanded Metal Companies, Feb. 20, 1902,* gives the following explanation of the fire-proof qualities of Portland cement concrete:

> The two principal sources from which cement concrete derives its capacity to resist fire and prevent its transference to steel are its *combined water and porosity*. Portland cement takes up in hardening a variable amount of water, depending on surrounding conditions. In a dense briquette of neat cement the combined water may reach 12%. A mixture of cement with three parts sand will take up water to the amount of about 18% of the cement contained. This water is chemically combined, and not given off at the boiling point. On heating, a part of the water goes off at about 500° Fahr., but the dehydration is not complete until 900° Fahr. is reached. This vaporization of water absorbs heat, and keeps the mass for a long time at comparatively low temperature. A steel beam or column embedded in concrete is thus cooled by the volatilization of water in the surrounding cement. The principle is the same as in the use of crystallized alum in the casings of fireproof safes; natural hydraulic cement is largely used in safes for the same purpose.
>
> The porosity of concrete also offers great resistance to the passage of heat. Air is a poor conductor, and it is well known that an air space is a most efficient protection against conduction. Porous substances, such as asbestos, mineral wool, etc., are always used as heat-insulating material. For the same reason cinder concrete, being highly porous, is a much better non-conductor than a dense concrete made of sand and gravel or stone, and has the added advantage of lightness. In a fire the outside of the concrete may reach a high temperature, but the heat only slowly and imperfectly penetrates the mass, and reaches the steel so gradually that it is carried off by the metal as fast as it is supplied.

***Cement*, May, 1902, p. 95.

CHAPTER XXII

SIDEWALKS AND BASEMENT FLOORS

The introduction of reliable American Portland cements has rendered concrete available for sidewalks and other similar purposes at a price not more than two-thirds of that previous to 1890, when German and English cements were used. Portland cement being thus commercially within reach of builders, masons have become familiar with its use, and concrete sidewalks, because of their economy and durability, are supplanting those of other materials.

Street pavements are also being made of concrete, and with apparent success,* by methods similar to those which obtain in sidewalk construction.

The essentials for a good concrete sidewalk are an artificial foundation of firm but porous material, through which the rain water may percolate, a base of good strong concrete, and a wearing surface of rich mortar, troweled to a smooth, dense surface. The walk must be divided into blocks, with the joints between them forming lines of weakness, so that if any cracks occur through shrinkage, settlement, or frost, they will occur at the joints and thus not be noticeable.

Vault light construction in concrete requires even greater skill than ordinary walks, and should never be attempted by inexperienced constructors.

The construction of basement floors is similar to sidewalk work except that in dry ground an artificial foundation is not always necessary, and, there being less danger of settlement and frost, the blocks of such a floor may be of larger size, having occasional joints to provide for contraction from changes in temperature.

Floors above the ground level in buildings whose design is considered in Chapter XXIII, page 451, may be surfaced with mortar in a manner similar to the wearing surface of walks, or the concrete may be floated without the extra coating of mortar.

MATERIALS FOR CONCRETE SIDEWALKS

The selection of a first-class Portland cement is an absolute necessity.† Natural cements will not stand the wear, and Puzzolan cements are liable

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**Engineering News*, Jan. 28, 1904, p. 84.

†See Cement Specifications, p. 29.

to surface deterioration from the action of the weather. Walks have been built with a Natural cement concrete base, and a wearing surface of Portland cement mortar, but the results have been unsatisfactory, for even if the surface coat is laid before the Natural cement concrete base has set, the Portland cement does not adhere strongly and is likely to scale off.

Mr. Harry T. Buttolph* suggests that the breaking up of the surface appears to be due to the difference in expansion of Natural and Portland cement. He has noticed that the surface of such slabs sometimes curls up like a sheet of paper.

For the foundation, by which is meant the prepared surface underneath the concrete, any porous material such as broken stone, gravel (preferably with sand screened out), or cinders may be employed.

For the base, which consists of a layer of concrete from 3 to 5 inches thick, ordinary materials, such as broken stone and sand, screened gravel and sand, or gravel as it comes from the bank without screening, may be used for the aggregate. Unscreened gravel is not generally advisable, however, because a more uniform mixture can be obtained by screening the gravel and remixing the sand with it in definite proportions. (See p. 112.) The proportions frequently used in our large cities for the concrete base are 1 part Portland cement to 2 parts sand to 5 parts stone, based in some localities upon the volume of cement as packed in the barrel, and in others upon the volume loose, although the resulting proportions obtained in the two cases are very different. (See p. 218.) In many cases these proportions are richer than is necessary. In Germany,† proportions 1:3:6 are recommended for heavy duty, and 1:5:10 for light work, while for ordinary requirements 1:4:8 are specified. The last two proportions appear rather lean for ordinary conditions, but 1:3:6, if the relative volumes are based on a unit of 3.8 cu. ft. to the barrel, should be satisfactory for ordinary conditions, with 1:2½:5 for more important construction, or for pavements to be subjected to severe usage, such as teaming. If the proportions are based upon the volume of cement measured loose, the required parts of sand and stone must be decreased by about 10%; thus 1:3:6 would become about 1:2¾:5½.

The wearing surface, whose thickness varies in different specifications from ½ to 1 inch, should be laid with the same first-class Portland cement as is the base. Customary proportions are equal parts of cement and aggregate. Either sand, or fine crushed rock, or a mixture of the two,

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*Personal correspondence.

†"How to Use Portland Cement," translated from the German of L. Golinelli by Spencer B. Newberry, p. 26.

may be used to form the mortar. If crushed rock is used, — and good crushed rock is usually preferable to sand, — it should be of a texture such as granite or trap, which will break into cubical, rather than flat or laminated fragments. The size of crushed stone specified by the majority of engineers is that which will pass a ¼-inch sieve, although a few cities require finer material, Chicago, for example, specifying* torpedo sand ranging from ⅛-inch down. Such sand is too fine to give a strong mortar. On the other hand, some cities, including Omaha, Neb.,† require crushed stone which will pass a ½-inch mesh sieve.

The requirements in various cities throughout the United States in 1900 are shown in the following table:

Requirements in Various Cities.‡ (*See p.* 437.)

City.	Foundation. Thickness.	Foundation. Material.	Base. Thickness.	Base. Proportions.	Wearing Surface. Thickness.	Wearing Surface. Proportions. Cement. Sand.	Dry Coating. Proportions. Cement. Sand.	Size of Blocks.	Guarantee.
	Inch		Inch		Inch				Yr.
Boston	12	Broken stone, gravel or cinders	3	1:2:5	1	1:1	...	3½ to 6 ft. sq.	10
Rochester ..	6	Sand, gravel, broken stone or cinders.....	||	1:5	1	2:3	...	....	3
Philadelphia	3	Sand, gravel, broken brick, stone or cinders	3	.....	2	1:2	1:1	....	..
Washington	0		4	1:2:5	1	2:3	1:1	....	5
Chicago ...	12§	Cinders..............	4¼	1:2:5	¾	1:1	...	5 ft. x 6 ft.	10
Milwaukee .	4	Cinders or broken stone	2½	1:3:5	1	1:1	...	24 to 36 sq. ft.	...
St. Louis...	8	Cinders..............	3½	1:3	½	1:1	...	....	1
Omaha	4	Gravel, slag or stone...	3	1:2:4	1	1:2	3:1	....	5

Coloring Matter. The appearance of a walk is improved by being slightly colored. The following formulas are recommended by P. B. Beery:¶

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*1899 Specifications.

†1898 Specifications.

‡From Typical Concrete Sidewalk Specifications, by Sanford E. Thompson, in *Cement*, July, 1900, p. 85.

§No foundation required where the soil is clean sand.

||Specified for each contract.

¶*Cement and Engineering News*, May, 1903, p. 85.

Black, use 2% Excelsior carbon black.
Red, use 10% best raw iron oxide.
Brown, use 6% best roasted iron oxide.
Buff, use 10% best ochre.
Blue, use 6% ultramarine.
White (or as near white as possible), use marble dust or white sand. Mr. Beery states also that Venetian red or lampblack should not be employed, as they fade, and that coloring should in all cases be made from the best metallic oxides, free from sulphur.

As a matter of fact, all colors will fade unless formed by the color of the crushed rock in the granolithic surface.

Quantity of Materials Required. The volumes of materials required to cover a certain area of surface are determined by the thickness of the walk or floor, the proportions in which the materials are mixed, and the character of the materials.

The following table gives the approximate quantity of materials necessary for 100 square feet of surface for walks of various thicknesses of base and wearing surface. It is assumed in compiling the table that the coarse aggregate of the base contains about 45% voids, and that the stone and

Materials for 100 Square Feet of Concrete Sidewalks. (See p. 438.)
Proportions based on a barrel unit of 3.8 cubic feet.

Base.							Wearing Surface.						
	Proportions. 1:2½:5			Proportions. 1:3:6				Proportions. 1:1		Proportions. 1:1½		Proportions. 1:2	
Thickness.	Cement.	Sand.	Stone.	Cement.	Sand.	Stone.	Thickness.	Cement.	Sand.	Cement.	Sand.	Cement.	Sand.
in.	bbl.	cu. yd.	cu. yd.	bbl.	cu. yd.	cu. yd.	in.	bbl.	cu. yd.	bbl.	cu.yd.	bbl.	cu.yd.
2½	1.10	0.39	0.78	0.94	0.40	0.80	½	0.85	0.12	0.68	0.14	0.56	0.16
3	1.33	0.47	0.94	1.13	0.48	0.96	¾	1.28	0.18	1.02	0.21	0.85	0.24
3½	1.55	0.55	1.10	1.32	0.56	1.12	1	1.70	0.24	1.36	0.29	1.13	0.32
4	1.77	0.63	1.25	1.51	0.64	1.28	1¼	2.13	0.30	1.70	0.36	1.41	0.40
4½	1.99	0.70	1.41	1.70	0.72	1.44	1½	2.56	0.36	2.04	0.43	1.69	0.47
5	2.21	0.78	1.56	1.89	0.80	1.60	2	3.41	0.48	2.72	0.57	2.26	0.63

NOTE.—Select and add together the quantities of each material corresponding to the required thickness and proportions of base and wearing surface.

sand are measured loose by shoveling into boxes or barrels, on the basis of the volume of a cement barrel of 3.8 cubic feet. For example, proportions 1: 3: 6 are equivalent to 1 barrel Portland cement, 11.4 cu. ft. of sand and 22.8 cu. ft. of broken stone or gravel, while proportions 1: 2 are equivalent to 1 barrel of Portland cement to 7.6 cu. ft., or one bag of Portland cement to 1. 9 cu. ft. of sand or crushed stone. The variation in volume of mortar produced with sand and crushed stone of different fineness may affect the quantities for wearing surface by at least 10%, but to provide for such variation, and to allow for waste, 10% has been added, in computing the values, to the quantities in the table on page 231.

Since the volumes are given separately for the base and wearing surface, the quantities required for walks of other thicknesses may be readily estimated, as illustrated in the following example:

Example: — What materials will be required for a walk 8 ft. in width and 150 ft. long, the base to be 3 in. thick, of concrete in proportions 1: 3: 6, and the wearing surface one inch thick, in proportions 1 part cement to 1 part sand?

Solution: — Referring to the table we find directly that for 100 sq. ft. of base 3 in. thick, 1.13 bbl. Portland cement, 0.48 cu. yd. sand, and 0.96 cu. yd. broken stone or gravel are required. Similarly, for 100 sq. ft. of the wearing surface one inch thick we should require 1.70 bbl. cement and 0.24 cu. yd. sand. For each 100 sq. ft. of completed walk there would therefore be needed 2.83 bbl. cement, 0.72 cu. yd. sand, and 0.96 cu. yd. broken stone or gravel; and since there are 1 200 sq. ft. in an area of 150 by 8 ft., for both base and wearing surface we should require 34 bbl. Portland cement, 9 cu. yd. sand, and 12 cu. yd. broken stone or gravel.

TOOLS

The following implements are required in ordinary concrete walk construction:

Mortar box for mixing the materials for wearing surface.

Platform about 12 ft. square for mixing concrete* (see Fig. 7, p. 22).

One or more iron wheelbarrows for handling the materials and the concrete (see Fig. 4, p. 18).

Square-pointed shovels (see Fig. 3, p. 18).

Hoe.

2-inch scantling of a width corresponding to the thickness of the walk.

⅞-inch stuff of same width as scantling, for curved forms.

Steel square.

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*Sometimes unnecessary.

Spirit level.

Straight-edge long enough to extend across the walk.

Two rammers about 5 inches square, with handles about 4 feet long (see Fig. 112, p. 373).

Wooden stakes.

Iron pins and twine for stretching line.

Mason's trowel.

Pointing trowel.

Plasterer's steel trowel (see Fig. 126, p. 443).

Plasterer's wood float.

Groover (see Fig. 127, p. 443).

Edging trowel (see Fig. 128, p. 444).

Dot roller (see Fig. 129, p. 444).

METHOD OF LAYING SIDEWALKS

Successful sidewalk construction is as dependent upon careful attention to small details which have been proved essential to good workmanship, as upon adherence to the more general directions given in any set of specifications. The full description of methods to be employed in laying a walk are given for the benefit of those who are unable to take advantage of the experience of specialists in this line. Experienced contractors often can perform such work better and cheaper than it can be done by day labor.

Thickness of Walk. A total thickness of 4 inches of concrete and mortar laid upon a 10-inch foundation of porous material gives excellent results for ordinary sidewalks, although 5 inches is often required for public works. In locations subject to wide changes in temperature, as Boston and vicinity, a thickness of 4 inches has proved satisfactory, while in some cities 3½ inches only is required. For a 4-inch walk it is advisable to make the base 3 or 3¼ inches and the wearing surface 1 or ¾ inch thick. The slope of surface often adopted is ¼ or ⅜ inches to the foot.

Driveways or walks which are subjected to excessive wear may be 5 or 6 inches thick, the upper 1 or 1½ inches constituting the wearing surface.

Foundation. The construction of the foundation is as important as the laying of the concrete. For out-of-door construction the foundation should generally be from 6 to 12 inches thick, depending upon the character of the soil. In localities unaffected by frost and having soil sufficiently porous to carry off surface water, the foundation may be omitted entirely, and the concrete laid upon natural ground excavated to the required depth.

In Washington, D. C.,* no foundation is specified, and even in Chicago* it is not required where the soil is clean, porous sand. For basement or cellar floors which are not to be subjected to frost, the concrete may usually be placed directly upon the soil; but in compact ground, or where surface water is troublesome, blind drains of pipe or of cobble stones, carefully rammed, should be laid at various points.

The materials for a foundation, where such is required, may be broken stone, gravel, cinders, or coarse sand. In order to make it more porous, broken stone or gravel should be screened. Whatever material is employed it must be thoroughly rammed so as to present a firm and unyielding surface. Cinders or sand should be thoroughly wet when being rammed.

Concrete Base of Walk. The coarse concrete constituting the main body of the walk is generally called the base. Before this coarse concrete of the base is placed, the surface must be carefully laid off into squares or blocks. Such divisions are absolutely essential, since the joints furnish lines of weakness along which cracks will occur if the concrete is affected by the freezing of the soil beneath tree-roots, unequal settlement, or temperature changes, and also facilitates the replacing of a block if one is injured from any cause.

There are three distinct methods of forming separate blocks: (*a*) laying the blocks alternately, and then filling in between them; (*b*) allowing the scantling of the forms to remain in place until after the concrete is laid, and then filling the spaces they occupied with lean mortar or sand; (*c*) placing tarred paper between the blocks. The first method is usually preferable.

The size of the blocks depends upon the width and shape of the walk or floor. Blocks nearly but not quite square have a better appearance than those which are distinctly oblong. The limit of size for a 4-inch walk is generally placed at 6 feet square. In 5-inch work this may be safely increased to 8 feet square. Joints should be placed around trees and about 6 inches from buildings, manholes, or other adjacent structures.

After ramming and leveling the foundation, if there is no curb to be formed, strips of scantling 2 inches thick, and of a width corresponding to the thickness of the walk, are placed on edge along the back and front lines of the walk, and held in place by stakes driven behind them. These strips should have notches cut in them to designate the location of the dividing line between the blocks. Other strips, located by these notches, are placed across the walk, which is now ready for the concrete.

The concrete materials in the specified proportions are mixed as de-

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*Specifications for 1899.

scribed on page 20. If the surface of the road is hard and smooth, the mixing may be done upon it without any platform. In any case, it must be very thorough, some contractors employing a man to rake each shovelful as it is turned by the two shovelers. Enough water should be added to produce a jelly-like consistency, the mortar rising to the surface when lightly rammed. The surface of the coarse concrete must be below the level of the top of the forms so as to give room for the finishing coat, or wearing surface.

If the walk or floor is laid in alternate blocks by the first method (*a*), described above, the forms around each block are left in until after the top coat or wearing surface has been placed, and has slightly stiffened, when they may be removed and the alternate blocks laid. The latter must be placed on the same day, however, to avoid difficulty in forming the surface joints between the stones. If a filler is placed between the blocks, the forms are lifted soon after the concrete of the base is laid, and before the wearing surface is spread, and the joints filled with sand or, in some cases, by a "separator" of lean mortar mixed, say, 1 part cement to 4 or 5 parts sand. Whatever the material used, it must be weaker than the concrete.

Wearing Surface. As soon as a few of the blocks of concrete base have been laid, and before they have set, the mortar for the wearing surface must be placed. This surface, as described on page 436, consists of a mixture of cement and sand, cement and fine crushed stone, or cement and a mixture of sand and stone. The materials should be very exactly proportioned, so as to give a uniform color. The cement must not be mixed with the sand long in advance of its use because the natural moisture in the sand will cake the cement. If the work is progressing so slowly that the cement must be measured by pailfuls, a determination must first be made of the number of pails of loose cement in a bag or barrel of packed cement, and the number of pails of sand in a barrel of loose sand, then the relative volumes calculated to allow for the increase in bulk of the loose over the packed cement. Each pail must be filled in exactly the same way, so that one measure will not be more densely packed than the next. The sand and cement must be mixed dry until the color is absolutely uniform, when, if coloring matter is used, it is added to this dry material. Water is added to give about the consistency employed by a mason in laying brick, so that it can be readily leveled off with a straight-edge. This mortar is carried from the mortar box to the walk in pails, and smoothed off with a straight-edge guided by the tops of the forms.

The surface is roughly floated with a plasterer's trowel, shown in Fig. 126,

soon after leveling with the straight-edge, but the final floating is not performed until the mortar has been in place from two to five hours and has partially set. The final floating is done first with a wooden float and afterwards with a metal float or plasterer's trowel. Just before the floating, a very thin layer of "dryer," consisting of dry cement and sand, mixed in proportions 1:1 or even richer, is frequently spread over the surface, but this is generally undesirable as it tends to make a glassy walk.

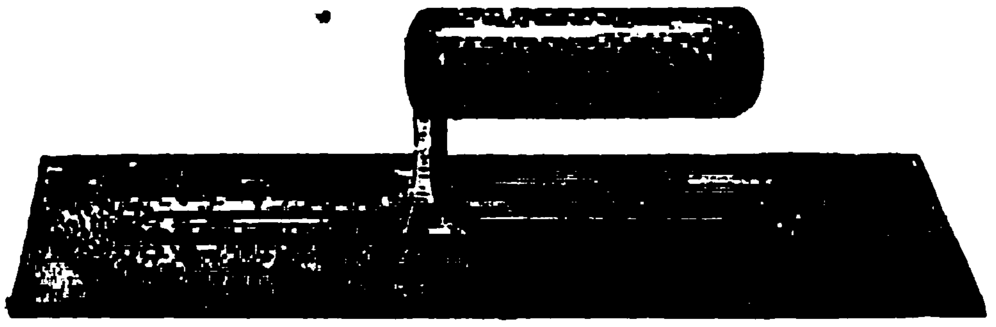

FIG. 126.—Plasterer's Trowel, or Metal Float. (*See p.* 442.)

The surface is now ready to groove, for by this time the intermediate stones should be in place. As has been stated, the cross joints are in line with notches in the outside forms. The mason can thus locate the joints between the blocks of base concrete. To find the line exactly, he runs his small pointing-trowel down through the upper layer, and feels for the joint below. With the ends of the joints thus marked, he lays a straight-edge flat across the walk against these marks, and, walking across on the straight-edge, marks the line and also cuts through the partially set mortar and concrete by running his small pointing-trowel to the full length of the blade. Moving the straight-edge back a fraction of an inch, he runs his groover (see Fig. 127) along the line cut by the trowel, using the straight-edge for a rule. Both edges of the walk are rounded off by the edging trowel (see Fig. 128), which is a small float with one of its edges curved. The entire surface is finally gone over once more with the metal float to erase any marks or scratches which may have been made. A dot roller (see Fig. 129) or grooved roller may be employed to relieve the smoothness.

FIG. 127.—Groover. (*See p.* 443.)

The exact time at which the surface should be floated depends upon the setting of the cement, and must be determined by the mason. Considerable skill is required in this troweling to prevent the formation of hair cracks by over-troweling, and to insure a surface which will not wear rough as a result of insufficient troweling.

If the walk is exposed to the hot sun it may be necessary to cover it with a wood or canvas frame, or with moist sand, for several days

after its completion, as it is absolutely necessary that it shall not dry out too quickly

Effect of Frost upon New Concrete Sidewalks. If concrete sidewalks are exposed to frost before thoroughly hard and dry, the surface is likely to blister and scale off in patches about $\frac{1}{16}$ inch thick. It is best, therefore, to avoid sidewalk construction in freezing weather.

Concrete Curbing. Concrete curbing for artificial sidewalks is largely displacing stone curbing. The curb is built just in advance of the walk. It is divided into blocks and is separated from the walk by joints similar to the joints between the blocks. The soil is excavated, and a foundation of porous materials of the same thickness as that employed under the walk proper is placed and rammed. In Boston* a layer of ordinary concrete 12 inches wide and 8 inches deep is placed upon this foundation to underlie the curb. The curb proper is 12 inches deep and 8 inches wide at the bottom, tapering on the outside to a width of 7 inches at the top, with its inside face vertical. At least one inch of the face and of the surface consists of mortar or granolithic, like the wearing surface of the walk. A typical sidewalk and curb is shown in Fig. 130. The back of the curb is formed against a temporary plank. For the face mold, a 12-inch planed plank is set on edge to the proper batter and may be held in place by driving stakes about 4 inches out from it, and nailing strips from the lop of these stakes to the top edge of the pank, so that they can be knocked up and the plank loosened without disturbing the face of the curb. When ready to place the concrete for the curb, which should be laid before the layer of concrete underlying it has set, a 1-inch board is placed on edge just inside of the 12-inch plank, with occasional thin strips or wedges between it and the plank. The coarse concrete of the curb is then placed back of this board, and thoroughly rammed so that its surface is one inch

FIG. 128.—Edging Trowel. (*See p.* 443.)

FIG. 129.—Dot Roller. (*See p.* 443.)

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*Specifications for 1899.

below the top of the forms, and when sufficiently hard, the 1-inch board is drawn up from the face, and with the aid of a trowel its place is filled with wearing surface material. The outside form is generally allowed to remain over night, and in the morning the outside surface is floated. A ruled joint like that between the blocks is formed between the curb and the remainder of the walk.

A metal corner is sometimes laid in the exposed edge of the curb to protect it from wear.

Combined Curb and Gutter. One of the advantages of a concrete walk lies in the ease with which it is adapted to special construction. A gutter 5 or 6 inches thick, with a pitch corresponding to the crown of the street, is often laid in combination with the curb. It is underlaid with a porous foundation, and in some cases by a sub-soil tile drain. The blocks forming the combined gutter and curb are made about 6 feet in length, and are in alternate sections so as to form definite cross joints, but each section of the curb and gutter must be built together, with no longitudinal joint between them.

FIG. 130.—Typical Concrete Sidewalk and Curb. (*See p.* 444.)

Vault Light Construction. Sidewalk lights over basement areas or subways are formed of circular lights of plate glass, set in reinforced concrete slabs, supported by steel or reinforced concrete beams. Steel rods about $\frac{4}{16}$ inch diameter are interlaced in both directions between all of the rows of glass discs. The width of the slab between beams is governed by the thickness of the slab, a customary width being 3 to 4 feet. The dimensions of the beams and girders, whether of steel or reinforced concrete, depend upon their loading and span. (See table, p. 313.) A typical vault

light construction supported by steel girders and stiffened by concrete ribs as designed by Mr. Ross F. Tucker, is illustrated in Fig. 131.

If concrete beams or stiffeners are used, they must be laid at the same time as the slabs are placed, so as to be in the same piece with them, but contraction joints must be provided as shown. In laying the slabs, the position of the glass discs may be located by an iron plate with holes of the size of the glass discs. On top of this iron form, a layer of oiled paper is spread to prevent the cement sticking to it, the lenses are set upon the paper over the holes, the reinforcing rods placed, and the mortar poured around the glass, and its surface troweled after partially setting, same as the surface of a granolithic walk. After the mortar has become thoroughly hard, the metal plate and the paper may be removed.

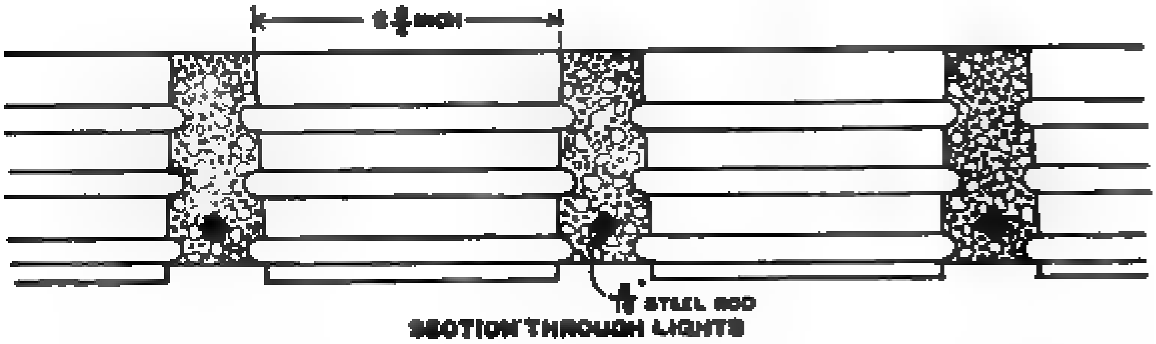

FIG. 131.—Typical Vault Light Construction. (*See p.* 444.)

COST AND TIME OF SIDEWALK CONSTRUCTION

The cost of concrete sidewalk or basement floor construction is extremely variable. The job at any one location is likely to be small, not occupying

more than a few days, so that the time and expense of transporting men and materials, and the time getting started upon the work, constitute an important item. The skill of the men employed in placing and finishing the concrete affects the cost still more, since an experienced gang may easily lay three times as much surface of walk in a day as inexperienced men, even if the latter are accustomed to ordinary concrete work. Excavation is another variable item, depending upon the quantity of earth to be removed and the character of the material.

A gang of convenient size consists of —

One mason.
One man to assist the mason in placing forms, and to level and ram the concrete.
Three men mixing and placing coarse concrete for base.
One man mixing top dressing for wearing surface.

If excavation is included in the work, more laborers may be needed. The amount of walk covered by a gang is limited by the surface which can be floated and troweled by the mason. Unless he works overtime, the laying of concrete must stop about the middle of the afternoon in order that the wearing surface may have opportunity to set. Meanwhile, the concrete gang may prepare and ram the foundation and get everything in readiness to begin concreting promptly the next morning. With a gang of the size suggested a foreman adds considerable to the expense, and it is often advantageous to so arrange the work as to make the mason responsible for its quantity and quality. A bonus paid for an excess over a certain area of surface covered is an effective incentive for a good day's work. In order to properly fix such a bonus the employer must know the relative times required for plain sidewalk and curb. The size of the blocks must also be considered, since the labor upon the joints forms a prominent division of the work.

Under average conditions a mason skilled in this class of work should float and trowel a surface of 600 to 700 square feet in eight hours, if no allowance is made for time which is necessarily lost between jobs and in commencing work. This lost time will lower the average by an amount varying with the size of the job. If the excavation is ready, five men working with the mason should prepare the foundation and place the base concrete and the mortar for the wearing surface for a walk 4 to 4½ inches thick. For a thicker walk, one more man may be required in the gang to keep up with the mason, since a thick walk requires more concrete or mortar.

The contract price for a granolithic or artificial walk from 4 to 5 inches

in thickness, with Portland cement at about $2.00 per barrel, varies from $0.22 to $0.30 per square foot. The cost of curbing runs about $0.75 to $1.00 per linear foot without a metal strip, and 25 to 50 cents higher with it.

DRIVEWAYS

For driveways the concrete is laid similarly to that in sidewalk construction. The total thickness may be 5 inches for light travel, or 6 to 7 inches for heavy teaming. Grooving the surface in 6-inch squares affords foothold for the horses.

CHAPTER XXIII

CONCRETE BUILDING CONSTRUCTION

The rapid development of the use of concrete both in the United States and Europe is the best evidence of its adaptability for a building material. This is exemplified in numerous structures which, not only from an engineering standpoint but architecturally as well, are models of the builder's art.

In work above ground, concrete is most extensively employed in the building of floors and roofs. Its especial availability for this class of construction has been made possible by the introduction of numerous systems of metal reinforcement, the application of which has resulted in the reduction of the thickness and brittleness of the slabs.

The fire-resisting qualities of Portland cement concrete when composed of first-class materials, such as sand, and gravel, hard broken stone, or cinders, appear both from experimental and actual fire tests to be equal or superior to those by any other material. (See Chapter XXI, p. 427.) Moreover, its strength and permanence, when it is carefully laid and properly reinforced, are unquestioned, and by employing a wet mixture the mortar in the concrete surrounds and effectually prevents the corrosion of the metal with which it is reinforced.

Its fire-resisting quality has led to the adoption of reinforced concrete for stairways, for columns and girders, and finally for entire buildings. The growing confidence in its utility for office buildings seems to promise for it successful competition with steel fireproof construction and a wide use in this class of structures. The cost of the reinforced concrete for an office building built of this material in 1904, based on actual construction records, with cement at $2.00 per barrel delivered on the work, was about 20% less than the estimated cost of the steel and tile of ordinary fireproof construction. As the concrete portion constituted about one-fifth of the total cost of the building, the net saving is reduced to about 4%, a very considerable sum, however, when figured on a fifteen-story office building. There is also an additional saving in other materials due to the reduction in height of the building because of the thin concrete floors, and to the fewer coats of plaster, with omission of furring, on walls and ceilings.

The Ingalls Building, designed by the Ferro-Concrete Construction Company and erected in Cincinnati, O., in 1903, was the first notable

example of a concrete office building in the United States. Sixteen stories high, it is entirely of concrete, with the exception of the facing of the exterior walls.

Although the expense of constructing forms presents an obstacle to the general use of reinforced concrete for factory buildings, in cases where contracting firms own the necessary appliances, this material may eventually prove cheaper than "slow-burning" mill construction with brick walls and timber beams and columns. In 1904, with Portland cement at $1.85 per barrel, including freight, the cost for reinforced concrete in representative factory buildings ran from 8% to 10% higher than the estimate for brick walls, timber columns and girders, and plank floors. As the concrete portion was only about one-half the total contract, the increased cost of the entire building was 4% to 5%. The concrete building has greater durability and is fireproof, thus affording lower insurance rates.

For dwellings and other small buildings the cost of the forms alone may exceed that of the materials and labor on the concrete. In estimating the labor, allowance must be made for the time which is often necessarily lost in waiting for the cement to harden or the forms to be removed. For these reasons it may be more economical to work with a small gang, taking an entire day to lay the concrete to the height of one section of forms.

For the cellar and foundation walls of frame or brick houses (see p. 461), concrete is frequently cheaper than rubble masonry.

A method of construction of light curtain or division walls consists in plastering Portland cement mortar upon metal lathing. A 2-inch wall thus made forms a permanent and fire-resisting partition. (See p. 469.)

Molded blocks of mortar or concrete (see p. 471) are adapted to certain classes of structures. Under favorable conditions the cost may be less than that of a brick wall of equivalent thickness.

CONCRETE FLOORS

Concrete floor slabs are supported by steel or sometimes by timber girders, or are formed in combination with reinforced concrete girders. The metal reinforcement which is universally adopted for the slab not only reduces the thickness and weight of the floor, but prevents sudden failure, an extremely important consideration in this class of structures.

Concrete floor panels between steel girders must compete chiefly with porous tiling and brick arches. The relative cost of these three materials, while dependent upon the location of the work and market prices, is usually, all things considered, in favor of concrete. The encasing of the steel I-beams with fine concrete or mortar affords fire protection to the girders and, if desired, a continuous surface for plastering.

Design of Concrete Floors. The thickness and the amount of reinforcement required for concrete slabs of different spans and loading may be taken from the tables on pages 317 or 318, or calculated by the formulas in Chapter XIV. The effect of the continuous area of a slab supported on all four sides is to largely increase the moment of resistance and consequently the load which may be carried. In practice, it is customary to allow one-fourth greater loading for floor slabs than for beams supported at the ends. The tables mentioned allow for this increase. The carrying of the haunches of the slab down to the lower flange of the I-beam (see Fig. 133, p. 458), in steel girder construction, also increases the strength of the slab.

The arrangement of the floor beams and girders in a building of reinforced concrete depends upon so many considerations that special study is required in each case.

The smallest quantity of material is required with floor panels of short span and frequent floor beams to support them. However, very thin slabs and beams of concrete are not easy to construct properly, and there is difficulty in imbedding the metal, so that we may, in general, limit the thickness of both to not less than 3 inches. For the slabs this minimum should be raised where a floor is liable to sudden strains, such as the falling of a load, which tend to punch a hole through the floor. For beams a more practical minimum width is usually 5 or 6 inches, since the cost of the form, which is but slightly more for a large than for a small beam, is a considerable item, and a deep, thin beam is in danger of buckling and requires frequent cross beams or stiffeners.

The spacing of the beams may, therefore, be governed in some cases by the required thickness of the floor slabs and in others by their own economical construction. Similar considerations, applied to column and foundation construction, govern the design of the principal girders.

The Ingalls Building* presents an example of slabs of long span supported by heavy girders, and the factory of the Pacific Coast Borax Company† an example of thin floor slabs with frequent deep but narrow concrete beams.

In simple cases the dimensions and reinforcement of concrete floor girders may be obtained directly from the table on page 302. More difficult problems require mathematical calculation, as treated in Chapter XIV. Not only must the size of the tension rods in the bottom of the beam be considered, but also the size and location of the U-bars, the reinforcement

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*See page 453.

†See page 463, also *Engineering Record*, July 30, 1898.

in the top of the beam, if required, and the proper connection with the columns. The girder illustrated in Fig. 132, page 455, is a typical design for a concrete beam supporting a heavy load, although the dimensions and reinforcement apply, of course, to a particular piece of construction.

There are several methods of laying floors supported by steel girders, one of the most common of which is illustrated in Fig. 133, page 458. The haunches of the slab are carried down to the lower flange of the I-beam, the under surface of which may be covered with metal lathing for fire protection and plastering. The I-beam may be entirely enclosed in the concrete, but it is difficult to place the material under the lower flange. Where head room is very valuable, the top of the slab is laid flush with the top of the beams and the metal is placed between the beams instead of running over them. In either case the outline of the concrete may form the ceiling, the plastering being placed directly upon it so as to form panels, or the ceiling may be suspended from the I-beams on metal lathing.

Floors are sometimes laid as continuous slabs, imbedding simply the upper flange of the I-beams in the concrete. The forms are cheaper to construct, but the strength is less than with the haunches, and the web of the I-beam is not protected from fire. For ceilings, separate slabs may be formed resting upon the lower flanges of the I-beams. Still another type of floor consists of concrete arches sprung between the lower flanges of the I-beams, just as brick arches are formed, and filled to the floor level with cinders. They do not necessarily require reinforcement.

The metal reinforcement in a floor slab should be as near to the under surface as is consistent with durability and fire resistance. For a strictly fireproof building it is safest to allow at least an inch of concrete below the metal, but under ordinary conditions this may be reduced to ¾ inch or ½ inch, provided the concrete is mixed wet and carefully placed around and under it. If plain rods are used, they must be prevented from slipping by selecting very long lengths or by anchoring the ends, or both. If the ends are bent for this purpose, there must be a considerable thickness of concrete beyond the bend to prevent the tendency under load to straighten out and thrust through the concrete.

Safe Floor Loads. The following loading for floors, suggested for the Boston building laws by a committee of the Boston Society of Civil Engineers in 1904, represents first-class modern practice:

All new or renewed floors shall be so constructed as to carry safely the weight to which the proposed use of the building will subject them, and every permit granted shall state for what purpose the building is designed

to be used; but the least capacity per superficial square foot, exclusive of materials, shall be:

For floors of dwellings and for apartment floors of apartment and public hotels, fifty pounds.

For office floors and for public rooms of apartment and public hotels, one hundred pounds.

For floors of retail stores and public buildings, except schoolhouses, one hundred and twenty-five pounds.

For floors of schoolhouses, other than floors of assembly rooms, eighty pounds, and for floors of assembly rooms, one hundred and twenty-five pounds.

For floors of drill rooms, dance halls and riding schools, two hundred pounds.

For floors of warehouses and mercantile buildings, at least two hundred and fifty pounds.

The loads for floors not included in this classification shall be determined by the Commissioner, subject to appeal, as provided by law.

The full floor load specified in this section shall be included in proportioning all parts of buildings designed for dwellings, hotels, schoolhouses, warehouses, or for heavy mercantile and manufacturing purposes. In other buildings, however, certain reductions may be allowed, as follows: In girders carrying more than 100 square feet of floor, the live load may be reduced by 10 per cent. In columns, piers, walls, and other parts carrying two floors, a reduction of 15 per cent of the total live load may be made; where three floors are carried, the total live load may be reduced by 20 per cent; four floors, 25 per cent; five floors, 30 per cent; six floors, 35 per cent; seven floors, 40 per cent; eight floors, 45 per cent; nine or more floors, 50 per cent.

Weight of Concrete in Floors and Girders. The following table is based on an average weight of broken stone or gravel concrete of 150 lb. per cubic foot, and of cinder concrete of 112 lb. per cubic foot, to each of which has been added the weight of 4 lb. per cubic foot to provide for maximum weight of about 1% of reinforcing steel.

The weight of stone concrete varies not only with the proportions of the mixture (see p. 244) but also with the specific gravity of the aggregate, and for particular cases, the weights on page 3, which are based on tests made at the Watertown Arsenal and Washington University and checked by calculation from the specific gravity of different materials, may be used instead of the table. The table, however, is sufficiently exact for ordinary practical purposes.

Floors in the Ingalls Building. In the Ingalls Building at Cincinnati, Ohio, whose floors above the second floor were designed for a live loading of 60 pounds per square foot, the principal panels, which are about 16 feet square, are 5 inches in thickness, and reinforced with ¾-inch rods. Smaller

panels of 3 to 6 feet in length are about 3 inches thick with ½-inch bars. The spacing of the rods varies with the length of the span. Where the panels are approximately square, the tension rods run in two directions, and where the panels are long and narrow, the tension rods run across the panel, with ¼-inch rods about 3 feet apart running lengthwise of the panel, to prevent contraction cracks. The principal girders are 32 feet long between centers of columns, and 27 inches in depth (measured to surface of concrete floor), and of width varying from 20 inches at the lower floors to 16 inches at the upper floors. Cross girders about 16 feet in length and 18 inches deep, of widths varying from 12 to 9 inches, are placed in the center of the span of the main girder, thus dividing the floor into slabs about 16 feet square. Fig. 132, page 455, is an isometric view showing the dimensions and reinforcement of the floor, main girder, cross girder, wall column, and wall in the fourth and fifth floors. The total distributed loading on the main girder is about 15 tons live load in addition to the weight of the reinforced concrete.

Weight of Reinforced Concrete in Slabs and Beams. (*See p.* 453.)

Weight of Reinforced Slabs per Square Foot.			Weight of Reinforced Beam one inch wide per foot of length.	
Thickness in.	Stone Concrete lb.	Cinder Concrete lb.	Depth of Beam in.	Stone Concrete* lb.
2	26	19	6	6.4
2½	32	24	7	7.5
3	38	29	8	8.6
3½	45	34	9	9.6
4	51	39	10	10.7
4½	58	43	12	12.8
5	64	48	14	15.0
5½	70	53	16	17.1
6	77	58	18	19.2
7	90	68	20	21.4
8	103	77	25	26.8
9	115	87	30	32.1
10	128	97	35	37.4

* Multiply by the length of beam in feet times its width in inches.

Materials for Floors. A first-class Portland cement which will meet the standard specifications given on page 29 must be selected. The rules for the selection of the aggregate are the same as for other classes of concrete. The size of the coarsest aggregate is often limited to one inch, but if well graded, so that the larger particles will not collect and prevent the flow of the mortar around the steel, the limit of size for beams, say,

5 inches in width and floors not less than 4 inches thick may be as high as 2 inches.

Cinders for concrete should contain but little unburned coal and be free from soot. A clean cinder will not discolor the palm when held in it and rubbed with the fingers. Usually a better mixture can be obtained by screening the fine stuff from the cinders, and then, if gritty, mixing it with sand, than by using unscreened material, although if the fine stuff is found by tests to be uniformly distributed through the mass, it may be used without screening and a smaller proportion of sand added.

Usual proportions for floor concrete are 1: 2½: 5, that is, one barrel packed Portland cement, 9.5 cu. ft. sand, and 19.0 cu. ft. of screened stone or screened cinders. If the thickness of the floor is such as to provide a wide margin of safety, the proportions may be 1: 3: 6 (based on a barrel of 3.8 cu. ft.), while for extra strong work 1: 2: 4 may be specified. For beams and girders 1: 2: 4 and 1: 2½: 5 are common proportions. Cinder concrete should not be used for girders, but under certain conditions may be employed for floor slabs. While it is lighter in weight, generally cheaper, and equal in fireproof qualities to first-class stone concrete (see p. 429), it is not so strong. Hence, for the same loading a greater thickness is required, and it is not usually economical even for floor slabs except the span and the loading are so small that the thickness of the floor is governed, not by required strength, but simply by the practical conditions of laying which limit it to a thickness of not less than 3 inches. In carefully designed reinforced buildings stone concrete is generally preferred.

The quantity of cement, sand, and stone or cinders required for any structure may be calculated from the table on page 231, or, for slabs, taken directly from the table on page 438.

Laying Floors. The general directions for mixing and placing concrete, given in Chapter II, p. 20, and Chapters XVI and XVII, are applicable to building construction.

The concrete must be mixed wetter than in sidewalk or basement floor construction, as described in the preceding chapter, so that the mortar may flow around the metal and thoroughly coat and protect it from rust and fire. The criterion of wetness may be that unless handled quickly it will flow off the shovel.

If the concrete floor is to provide a wearing surface, a granolithic finish may be given to it by laying a mortar wearing surface before the lower portion has set, as described for sidewalks in the preceding chapter, or the concrete may be troweled without the coating of mortar. The latter plan is amply sufficient for floors which are not subjected to excessive wear.

For a board floor, nailing strips are laid upon the concrete, or imbedded in it at right angles to the supporting beams. With cinder concrete the plan is sometimes followed of nailing the floor boards directly into the concrete. The objection to this is that the surface of the concrete must be leveled with great care, and it is difficult to relay the boards if a new floor is required.

The cost of the labor of laying a concrete floor is dependent upon the character of the building. In a case under the observation of the authors, where the floors consisted of cinder concrete resting upon steel I-beams, a gang of nine laborers, with a foreman (in addition to the engineman, who ran the elevator), mixed concrete in the basement to supply a gang of eleven men, with foreman, who, on one of the upper floors, were placing metal, wheeling concrete, leveling it, and cleaning forms. Six carpenters, with foreman, were employed building the forms, which were supported from the girders, in advance of the concreters. This gang averaged 22 to 25 batches (corresponding to 17 to 19 cu. yd.) of 1: 2: 5½ cinder concrete in nine hours.

Fig. 133. — Form for Concrete Floor between Steel I-Beams. (*See p.* 458.)

Floor Forms. In a large building the floor panels should if possible be so designed that the same forms may be used more than once, although they must not be removed until the concrete has attained sufficient strength to sustain its own weight and any loading which will come upon it.

If the floor slabs are supported by steel I-beams, the forms may be attached to the lower flanges, as shown in Fig. 133, a design of Mr. William F. Kearns.

If the girders are also of concrete, the supports for the form must be heavy enough to carry the weight of the beam of concrete, as well as the floor slab and the men and materials upon it. The forms must be so tight as to prevent the water and thin mortar running away from the concrete and carrying off the cement. This may best be accomplished by tongued-and-grooved or bevel-edged boards, but it is often possible to use square-edged lumber if it is thoroughly wet to swell it before placing the concrete. Joints in the beam forms may be covered with cleats.

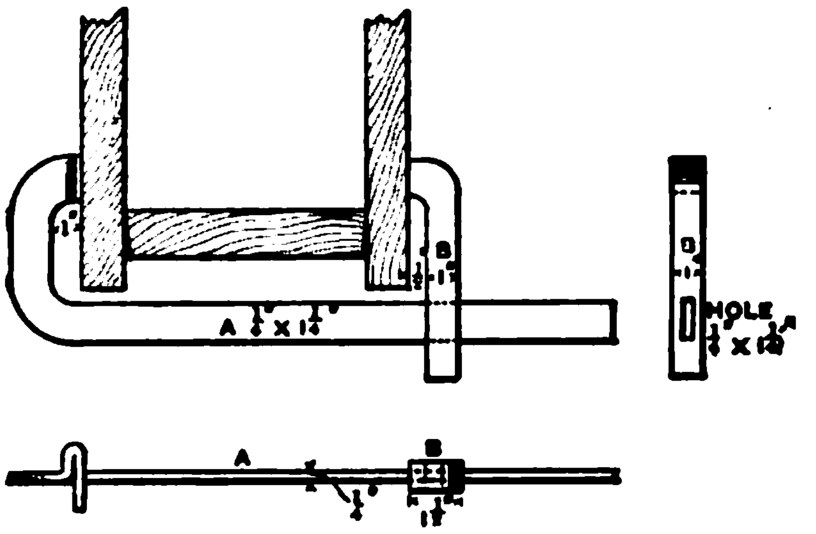

FIG. 134. — Clamp for Beam or Small Column Form. (*See p.* 459.)

A simple form of clamp for beam or small column forms, used originally in Europe, is shown in Fig. 134. The hook, *A*, is a plain piece of flat iron ¼ inch by 1½ inches, with one end bent and curved as shown. The dog, *B*, is a square piece of iron, with the end slightly turned and a hole slightly larger than the flat iron, *A*, punched through it. This is tightened by hammering on its lower end. The outward pressure of the form boards upon its upper end causes it to bind, and prevents it from slipping back. If it fails to hold, in any case, a wooden wedge is readily driven in to assist in tightening.

CONCRETE STAIRS

The design of concrete stairs is a simple problem in reinforced concrete construction. A stairway may consist (1) of an inclined slab of reinforced concrete with the steps molded upon its upper surface, or (2) of two or, for a wide stairway, three inclined girders to form the stringers, with the stairs between them. The first method is suitable for short flights not over 8 or 10 feet in length measured on the slope, and the thickness and reinforcement are calculated as for a slab supported at the ends. (See pp. 316 to 319.) The principal reinforcement is of course in the direction of the length with occasional cross metal for stiffening. A slab 5 inches thick measured at the foot of the risers is suitable for a stairway half a story high.

When built with side girders, the dimensions of each of the latter may be calculated as a concrete beam with a longitudinal rod near the lower surface. A small rod also runs across from girder to girder at the foot of each riser so that the risers are practically reinforced beams. It is usually cheaper to construct the under side of the stairs as a slab than to build

forms for each stair. The forms for the stringers may consist of planks notched for treads and risers, with boards nailed across as molds for the faces. If a fine finish is desired, the method of surfacing described for curbing may be followed. (See p. 444.)

CONCRETE ROOFS

Concrete roofs are designed and laid in much the same manner as are floors. The forms also are similarly constructed. As the weight of the roof itself forms a large proportion of the total load upon the girders, cinder concrete, because of its light weight, is especially adapted to this class of construction. The strength of the concrete may also play a smaller part in roofs than in floors, because the length of span may be governed by other conditions, and the concrete may often be laid as thin as is practicable to lay it and properly imbed the metal.

The wetness of the concrete is limited by the slope of the roof, although for a steep slope it may be necessary to confine the surface of the concrete by forms.

The proper thicknesses and reinforcement for different spans may be obtained from tables on page 317 or 318, selecting the weights from the data in the paragraphs which follow.

Roof Loads. A roof load is made up of the weights of the roof itself, the roof covering, the snow load, and the wind load.

The weight of the concrete may be obtained from the tables mentioned.

Prof. Mansfield Merriman* gives the following estimates for the weight of roof covering:

Tin, 1 lb. per square foot of roof surface.
Iron, 1 to 3 lb. per square foot of roof surface.
Slate, 10 lb. per square foot of roof surface.
Tiles, 12 to 25 lb. per square foot of roof surface.
Average may be taken at 12 lb. per square foot.

The snow load varies with the slope of the roof and the locality. Prof. Merriman allows for an approximate average 15 lb. per square foot of horizontal area.

The wind load, which acts horizontally, varies with the velocity of the wind, a usual pressure being assumed as 40 lb. per square foot of vertical surface. This pressure multiplied by the sine of the angle of slope of the roof gives the pressure normal to the surface.

Special Roof Construction. Concrete is well adapted for arch roofs,

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*Merriman's Roofs and Bridges, page 4.

groined arches, and domes. The dome of one of the Bi-Centennial Buildings at Yale University, New Haven, Conn., for example, 55 feet in diameter at the bottom and 34 feet high, consists of a skeleton of 24 8-inch I-beam ribs, supported at the top against a circular steel rim, with reinforcing metal imbedded in the 3½-inch thickness of concrete between them. The surface of the concrete was formed by "screeding" it with a curved templet whose length was the entire height of the arch.

CONCRETE WALLS

If Portland cement concrete could be laid in thin walls as cheaply as in mass work it would be one of the most inexpensive materials for permanent construction. As a matter of fact, an experienced contractor can build a 6-inch wall of concrete which will be stronger, more durable, and no more expensive than a 12-inch wall of brick.

The chief cost in concrete wall construction is in the labor of building and raising the forms and of hoisting the concrete. The former varies with the method of construction and the number of angles in the wall. In the case of a large structure the concrete may be hoisted in elevator buckets* by power. If the building is small and the concrete is hauled up by hand in buckets to a height of, say, 15 feet, at least twice as many men will be required to fill pails, haul up, and carry to place as are needed for measuring and mixing the concrete on the platform below.

Methods of surfacing concrete walls are described on page 380. Plastering is unsatisfactory.

Cellar Walls. Cellar or basement walls adapted to withstand earth pressure may be thinner when of concrete than when built of stone, because laid as a continuous vertical slab supported at top and bottom.

For a wall of 1:3:6 Portland cement concrete with a spreading base imbedded in the earth, a thickness of 10 inches will withstand without reinforcing metal a pressure of 6 feet of earth. If the top of the wall is strengthened by a wooden sill imbedded in or dogged to the concrete, and the sill is stiffened by floor joists, the wall becomes a slab supported at its bottom by the earth and at its top by the sill. A 6-inch wall 8 feet high will thus withstand the pressure against it of 6 feet of earth. However, ¼-inch rods, spaced about 2 feet apart in both directions, will greatly stiffen so thin a wall, and prevent cracks before the concrete is thoroughly hard. If desired, a coping of concrete wider than the wall itself may be formed at the top and a ½-inch rod placed horizontally in its inner face.

*Method used at the Ingalls Building is illustrated in *Engineering News*, July 30, 1903, p. 95.

The earth must not be filled in against the back of the wall until three or four weeks after placing, unless portions of the interior forms are left in place and carefully braced.

Designs for reinforced concrete retaining walls are illustrated on page 491.

A simple form for a cellar or foundation wall is illustrated in Fig. 135. A ranger, *AA*, is lined, and lightly spiked to occasional studs whose pointed ends are driven into the ground, and kept in line by strips of wood running from it to stakes in the bank. In some cases it may be advisable also to set a lower ranger between the studs and the bank. Occasional stakes, *BB*, are driven in the ground, and a ranger, *CC*, for the inside row of studs, is laid on top of them, lined, and lightly spiked to them, while the upper ends of these studs are held by cleats, *DD*, run across to the inner row of studs. Vertical strips, *EE*, about ⅞ inch square, are placed inside of each stud for the form planks to rest against, and after a section of concrete is laid are easily knocked out, and the form planks raised to another level. The first layer of concrete is allowed to flow out under the lower plank to form a footing, above which the cellar floor is laid. The number of the laborers and the height of the forms should be such that the planks may be raised each morning, provided the concrete is hard enough to withstand the pressure of the thumb without indenting.

FIG. 135.—Form for Cellar Wall. (*See p. 462.*)

Walls for Buildings. Concrete walls are either of single thickness, or double with an air space between. The double wall has greater stability, and the air space renders the interior of the building less subject to changes in temperature and more completely moisture-proof. Moisture is likely to collect on the inside of a single wall.

A single concrete wall 4 inches thick with its base spread to provide a footing is at least equivalent to an 8-inch brick wall, and a 6-inch concrete is at least equivalent to 12 inches of brick. It is advisable to place small reinforcing rods, about ¼ inch in diameter, 18 inches or 2 feet apart in walls 6 inches thick or under, not only to increase their permanent strength, but to guard against accidents during or immediately after construction. Occasional projections or pilasters improve the appearance and add to the strength of a single wall.

Each face of a hollow wall is usually 3 to 4 inches thick, 3 or 3½ inches being the minimum thickness at which concrete can conveniently be placed.

The four-story factory building of the Pacific Coast Borax Company at Bayonne, N. J., designed by Mr. E. L. Ransome, is an excellent example of hollow wall construction. The thickness of both faces of the walls is 3½ inches. The walls of the first story are 16 inches from surface to surface, that is, the space between is 9 inches, while the walls of the upper stories are made thinner by reducing the width of the hollow space. The general construction of a hollow wall is illustrated in Fig. 137, page 465.

The walls of the Ingalls Building consist of concrete 8 inches in thickness, faced with brick or marble. They are supported by reinforced columns spaced about 16 feet on centers, and the portions of the wall at the floor lines, that is, between the top of the window of one story and the window-sill of the story above, are, in reality, concrete beams reinforced by two ½-inch bars placed 2 inches above the top of each tier of windows, with ¼-inch horizontal bars 2 feet apart over the remainder of the wall. In addition to the column reinforcement vertical bars are placed 2 inches from each window opening.

The marble facing is supported at each floor line by triangular projections in the concrete, and the brickwork in the stories above by square projections 3½ inches wide. The marble is also held at each horizontal joint by anchor bolts imbedded in the concrete, and the brickwork by ties of round, straight rods about 8 or 9 inches long and ⅛ inch in diameter, placed through small holes in the outer forms before concreting so as to extend 5 inches into the concrete.

Wall Forms. A simple form for a cellar wall is illustrated and described

on page 462. A form for a wall of single thickness is illustrated in Fig. 136. The concrete is first laid to the full height of the ribs, then the bolts are loosened, the ribs raised one-half their length, so that one-half of each still laps over the concrete to keep the wall true and straight, and the forms are again filled with concrete to the top. Two bolts to each pair of ribs are all that are required after the concreting is commenced. These are removed before the wall is hard, so that they need be simply greased and the holes filled solid full with mortar mixed in the same proportions as the mortar in the concrete. The collar and set screw shown in detail is convenient where the walls or columns are of various dimensions, although usually an ordinary threaded bolt with nut and washer may be used.

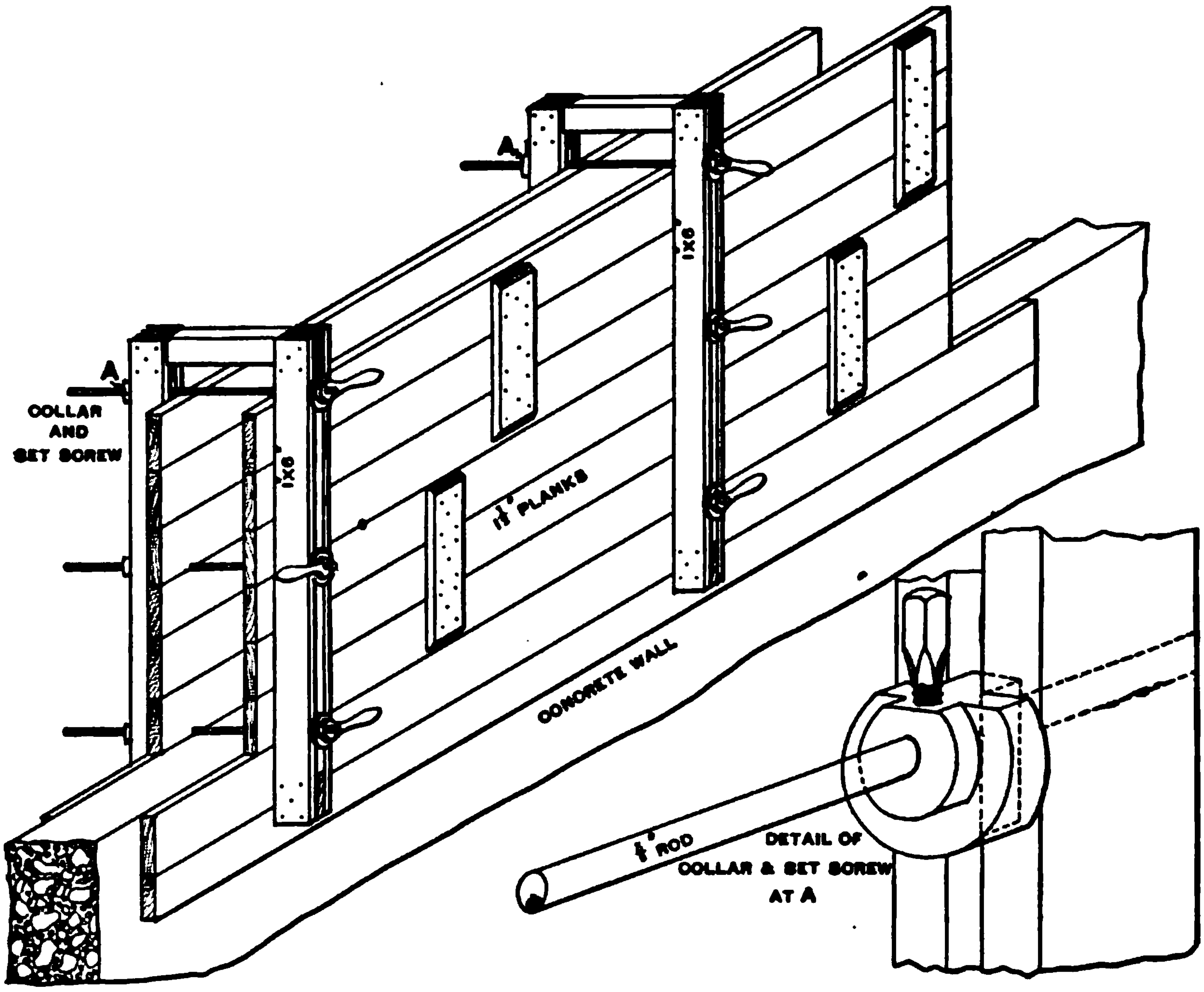

FIG. 136. — Ribs for Holding Form Plank. (*See p.* 464.)

A design for a form for a hollow wall is shown in Fig. 137. The ribs and bolts are so arranged that the latter do not pass through the concrete, the form being raised when the concrete reaches their level. In the same figure is shown a style of tongued and grooved molding with edges slightly beveled, which may be used to form the horizontal joint instead of nailing

a triangular strip upon the planks. If the surface is finished as a monolith of course no moldings are required. The forms must be nearly water-tight, to prevent the mortar running away from the stones.

Placing Concrete in Walls. For thin walls it is necessary to use mushy concrete, so soft that it must be handled quickly or it will run off the shovel. It should not, however, be so wet that the mortar is watery, or it will run away from the stones and leave pockets in the finished work. The concrete should be joggled rather than rammed, the chief object

able voids on the surface. The ramming of concrete is discussed on page 373, and methods of surfacing are described on page 380.

The size of stone for walls is sometimes limited to ¾ inch or one inch. However, a larger sized material, even up to 2 inches, has been used by Mr. Thompson in 4 and 6-inch walls with satisfactory results.

CONCRETE COLUMNS

For concrete columns in proportions 1 : 2½ : 5, conservative designers allow from 250 to 350 pounds per square inch (17½ to 24½ tons per square

foot). For ordinary concrete this allows a factor of safety of about 9 to 6 at one month, or 12 to 8½ at six months. Small columns, under, say, 2 feet in area, should receive rather less loading, perhaps 50 pounds per square inch less, than larger columns, because of the greater difficulty in placing the concrete. Small specimens of concrete usually show lower compressive strength than large specimens. (See p. 279.)

Columns are reinforced by various methods: (1) Small diameter rods are placed in the corners of the columns, with occasional horizontal hoops surrounding them; (2) large rods are placed in the interior of the columns to assist the concrete in taking the vertical compressive stress, and (3) some form of spiral or a succession of hoops is employed to utilize the increase in strength of hooped concrete over plain concrete. The first method, which is often used in combination with the other two, adds slightly to the actual compressive strength of a column, and assists in preventing sudden failure before the concrete is thoroughly hard.

The columns of the Harvard Stadium*, illustrated in our frontispiece in process of construction, range in size from 14 inches square to 24 by 33 inches, and contain ⅜ and ½-inch rods in the corners with square loops of ¼-inch rods placed around them horizontally at intervals of about fifty times the diameters of the loop rods. The allowable compressive stress for 1:3:6 concrete in columns was taken at 350 lb. per square inch. The outer wall is supported by hollow piers, 66 by 36 inches over all, 4 inches thick on the longer faces, and 6 to 8 inches thick on the ends.

The 1904 specifications of the Prussian Public Works place the horizontal rods at distances apart of not more than thirty times their diameters.

A typical section of column in the Ingalls Building is shown in Fig. 132, page 455. The rods designed to assist in bearing the compressive stress are 4 inches in diameter in the lower portion of the column, and are gradually reduced to one inch diameter at the upper stories. They are connected at the ends with pipe couplings and the joints grouted. The outer rods on each edge of the column are designed to resist the wind stresses. To avoid complication in the drawing, these are not shown at the floor level. Formulas for reinforced columns are given on page 329.

The flexure of the column is disregarded by the Prussian regulations until the height of the column exceeds eighteen times its least diameter. Some other authorities limit the height to twelve times the least diameter before taking account of this in the calculation. In cases where the loading is eccentric, however, computation should be made even for short columns, as outlined on page 253.

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*Described by Lewis J. Johnson in Journal Association Engineering Societies, June, 1904, p. 293.

FIG. 138. — Molds for Columns at Harvard Stadium. (*See p.* 469.)

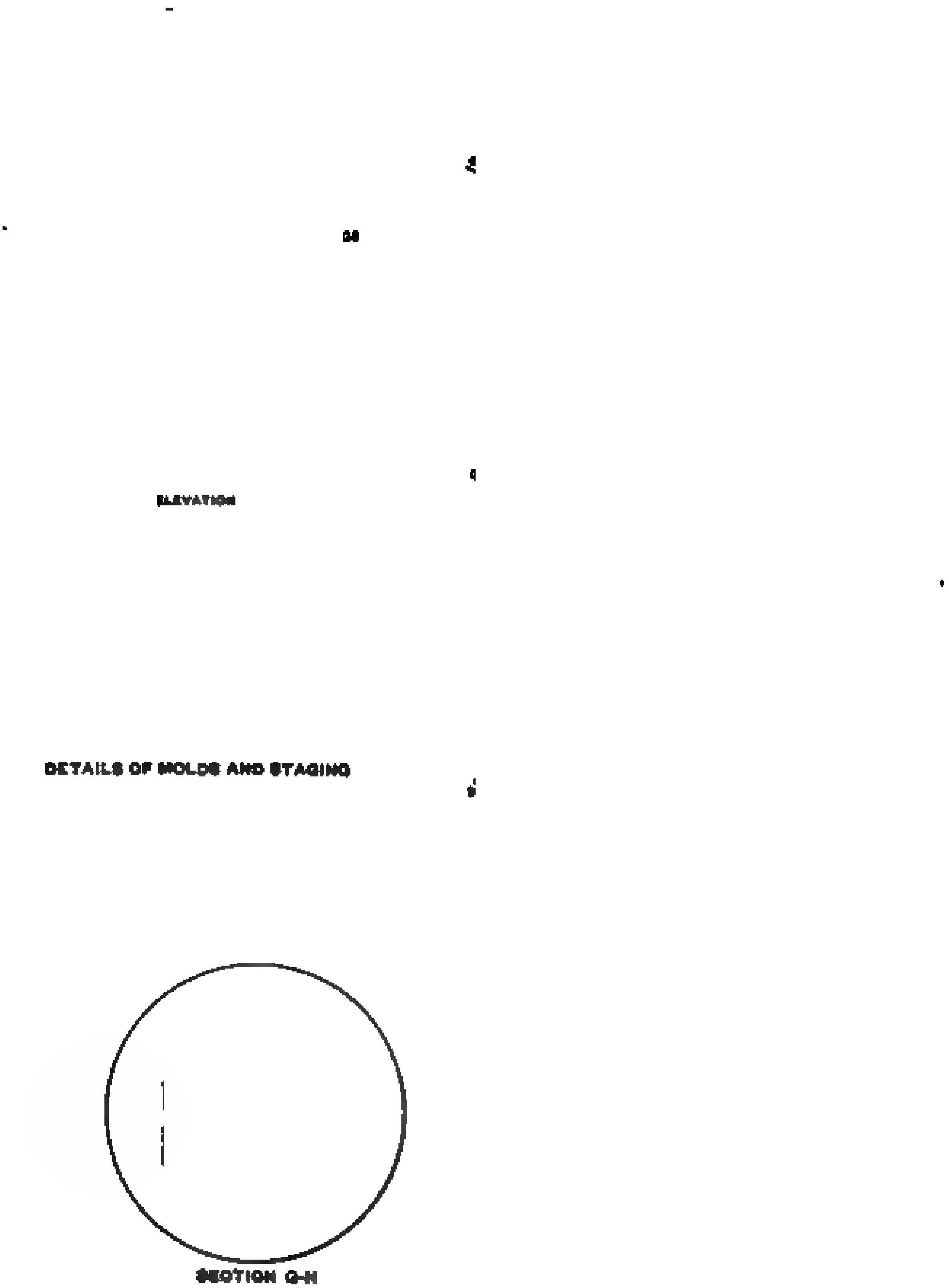

Fig. 139.— Chimney of Pacific Electric Railway Power House. (*See p.* 469.)

The construction of the molds for a concrete column is illustrated in Fig. 138, which shows a column of the Harvard Stadium under construction.

CONCRETE CHIMNEYS

High chimneys have been built of concrete in different parts of the United States. One of the most notable of these is the chimney of the Pacific Electric Railway Power House,* built in 1902. A sectional elevation, together with elevation and plan of molds and interior staging, are shown in Fig. 139. The two shells are entirely independent of each other, the vertical ribs, carried up the entire height of 180 feet, being formed so as to give a clearance between their points of ¾ of an inch. The forms were supported by scaffolding within the flue, where the elevator was also placed. The inner shell was kept above the outer one, and a height of 5 feet of concrete laid in each shell in an afternoon, so that by nine o'clock next morning the concrete was hard enough to permit raising the forms. The forms were 7 feet high, and the alignment was maintained by clamping the lower 2 feet by turnbuckles upon the concrete laid the day before. The chimney is the design of Mr. Carl Leonardt.

WALLS OF MORTAR PLASTERED UPON METAL LATH

Partitions of plaster from metal lathing are used extensively for fireproof office buildings and hotels, and are also adapted, when made with Portland cement mortar, to certain classes of outside walls.

For a one-story building, timber or steel posts may be set upon concrete foundations, and the walls constructed by using ¾-inch or 1-inch channel irons for studding, to which the metal lathing is attached, and then covered (on both sides) with Portland cement mortar about 2 inches thick, the studding being generally set from 12 to 16 inches on centers, the spacing depending on the height of wall. Such walls are also adapted for high buildings where steel frames are used, as the studding can be securely bolted to the steel work, and the metal lathing and cement applied in the same manner as for one-story buildings.

For curtain walls the first coat of mortar is usually mixed with one barrel of first-class Portland cement to three barrels of coarse sand, and one cask of lime putty, or paste, into which is mixed a small quantity of long cattle hair. The second coat, which is applied before the first coat is thoroughly dry, consists of one barrel of Portland cement to three barrels of sand with about a bucketful of lime putty, without hair. The finish coat is generally mixed in the proportions of one part Portland cement to two parts sand.

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*For full description see *Engineering News*, April 2, 1903, p. 308.

This finish coat may be troweled or floated to a smooth or rough surface, as may be desired, or it may be given what is known as a "slap-dash" finish by throwing the mortar on with a brush or twig broom.

Ornamental Construction. Concrete or mortar may be cast by special molds into blocks of any desired size or shape, or molded for ornamental decoration in designs which vie both architecturally and in durability with finely carved sandstone, limestone, and granite. The color may be slightly varied by mixing different kinds of crushed stone. Artificial coloring matter is apt to fade.

Fig. 140. — Pouring Seat Slab of Harvard Stadium. (*See p.* 470.)

Ornaments are run whole in a mold which is made in halves, or are molded in two or three pieces and cemented together. Molds of plaster-of-Paris, shellacked within, are commonly employed.

Another method of molding, similar to that employed for iron castings, is with fine, damp sand, which is sometimes treated by a patented process. A wooden core is made and sand packed around it, then the core is removed, and the mortar is poured in. The surfaces, after setting, may be rubbed down and floated. Fig. 140 illustrates the pouring of a seat slab

at the Harvard Stadium.* The wooden core, which was of the form of an L, for riser and tread, has been removed from the sand, reinforcing wire placed, and thick grout of the consistency of cream is being run in from a box car. The proportions of material were about one part Portland cement to 2½ parts fine crushed trap rock under ⅜-inch diameter.

CONCRETE BUILDING BLOCKS

Numerous machines and patented methods are on the market for forming building blocks of Portland cement mortar or concrete to compete with brick and stone for house fronts. Some of the machines form the blocks from concrete mixed rather dry and pressed into the mold, while other methods employ a semi-liquid consistency, and the material is merely poured into the molds. The blocks may be hollow so as to extend clear through the wall, or each face of the wall may be laid with separate blocks.

If care is exercised in molding and the sizes and surface appearance of the blocks are varied, a wall of pleasing architectural effect is possible.

The material for building blocks should be first-class Portland cement and fine crushed rock, or fine gravel and sand ranging in size from ½ inch in diameter to dust. Fine sand or fine dust alone makes with Portland cement a very porous stone, and must therefore never be used.

*Lewis J. Johnson in Journal Association Engineering Societies, June, 1904, p. 305.

CHAPTER XXIV

FOUNDATIONS AND PIERS

Concrete excels as a material for foundations, and here finds its widest and most important field of usefulness. It is pre-eminently adapted to such construction, because the stresses are chiefly compressive, the forms are easily built, and the surface appearance need not be considered.

Concrete is peculiarly suited to under-water foundations because, although it requires careful handling, it can be placed with great facility. It is now used even in piling. (See p. 477.)

Within recent years concrete has been adopted for foundations above ground, such as bridge piers, and is standing the test of durability even when subjected to excessive wear and impact. (See p. 483.)

Since the design of a foundation or sub-structure is governed almost as much by the character of the underlying rock or soil as by the superstructure, brief reference is made to the standard practice in estimating loads, although the treatment of engineering principles, as such, is not within the province of this treatise.

BEARING POWER OF SOILS AND ROCK

Sound hard ledge will support the weight of any foundation and superstructure, but if the rock is seamy or rotten it may require thorough examination and special treatment. If its surface is weathered, it must be removed. A sloping surface must be stepped or the foundation designed with sufficient toe to prevent sliding.

The sustaining power of earths depends upon their composition, the amount of water which they contain or are likely to receive, and the degree to which they are confined. An approximate idea of the loads which may be safely placed upon uniform strata of considerable thickness is given by Mr. George B. Francis*:

> There are several classes of strata that are readily definable, such as ledge rock, hard pan, gravel, clean sand, dry clay, wet clay, and loam, and when these strata are of considerable thickness and uniform for considerable areas, they may be loaded with safety (provided the material

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*Journal Association Engineering Societies, June 1903, p. 340.

placed thereon is not of less density than the natural material upon which it is placed, viz., concrete or brick work on ledge rock) as follows

Ledge rock, 36 tons per square foot.
Hard pan, 8 tons per square foot.
Gravel, 5 tons per square foot.
Clean sand, 4 tons per square foot.
Dry clay, 3 tons per square foot.
Wet clay, 2 tons per square foot.
Loam, 1 ton per square foot.

Mr. Francis, however, calls attention to the many kinds and mixtures of materials, and to the consequent impossibility of applying such specific rules as the above to all cases. He also emphasizes the necessity for varied and ample experience when fixing safe allowable pressures.

If the piles are driven to firm strata, such as rock or hard pan, the loading which a pile will stand is determined by the crushing strength of the timber. If supported wholly or in part by friction, it is customary to calculate the safe loading by a formula based upon factors obtained by experiment, or by one based upon the penetration of the pile from the blow of the pile driver.

An engineer experienced in pile driving in a particular locality can often determine by judgment whether the piles have reached a firm bearing, but it is usually safer to formulate exact specifications. Mr. Joseph R. Worcester* advises for piles which meet a hard resistance, a penetration of one inch under a 2 000-lb. hammer falling 10 feet, and for piles which hold by friction, a penetration of 3 inches under a 2 000-lb. hammer falling 15 feet. He prefers heavier hammers if they are available.

A mean of the various formulas† gives for approximate average values, after applying a factor of safety of 3, a safe load of 16 tons for bearing piles and 9 tons for friction piles.* These loads apply to ordinary piles of spruce and Norway pine.

A commonly used formula for determining safe loading on piles with reference to the penetration under blows of the hammer is the *Engineering News* formula, which is as follows:

Let

P = safe load in tons upon a pile.
W = weight of hammer in tons.
h = height of fall in feet.
p = penetration in inches under last blow.

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*Journal Association Engineering Societies, June, 1903, p. 285.

†The various pile formulas are tabulated and discussed by Ernest P. Goodrich, in Transactions American Society of Civil Engineers, Vol. XLVIII, p. 180.

Then

$$P=\frac{2Wh}{p+1}$$

Mr. Worcester states with reference to spacing piles:

> The minimum distance between centers of piles depends upon two factors: the hardness of the soil and the size of the butts. Ordinary spruce piles may be well driven 24 inches on centers, while large and long piles cannot be driven to advantage closer than 30 inches. Another governing condition must be taken into account, however, and that is the supporting power of the soil as a whole. Where the piles reach a real hard pan, the soil will generally resist all the pressure that the piles can bring on it, unless it consists of a thin crust overlying a soft material; but when the soil is so soft that the piles hold by friction only, and there is enough friction to carry all the soil between the piles down with them, in case they go together, the spacing becomes a question of how much the underlying soil will support per square foot. For example, if the soil can only support 2 tons per square foot, and the piles could each carry 18 tons, it is useless to place them closer than 3 feet on centers.

CONCRETE CAPPING FOR PILES

Although some authorities advocate stone capping for piles, even if the cost is more, it is generally considered good practise to lay the concrete directly upon the head of the pile. The ground is excavated to a depth of one or two feet around the piles, and if very soft, a layer of broken stone or chips may be spread and rammed hard upon it before laying the concrete. The load is distributed by the concrete, and the supporting power of the soil between the piles is utilized.

The thickness of the concrete above the piles must be sufficient to distribute the superimposed weight, and the reactionary load of the pile head acting upwards. If the layer is very thin there may be danger of the pile head shearing through the concrete. The objection sometimes raised to concrete capping is that the upward crushing stress upon the concrete by the head of the pile may be excessive, especially if loaded before the concrete is thoroughly hard. In considering this tendency, it must be borne in mind that under concentrated loading the concrete will sustain a higher stress per unit of area of contact than if the load is distributed. (See p. 250.)

DESIGN OF CONCRETE FOUNDATIONS AND FOOTINGS

The load upon a building foundation need not always be taken as the dead load plus the entire live load for which the superstructure is de-

signed, because in most structures the full live load will never be imposed upon all the floors at the same time. A conservative suggestion for reduction in the live load is given on page 453.

Eccentric pressure caused by an excessive load on a particular part of a foundation must be provided for, or unequal settlement may occur. In many classes of construction it is desirable to separate the foundations under different columns, and give each column the area required to withstand its own load, instead of making one continuous mass, and, furthermore, to arrange for unit pressures on all foundations of a structure, so that the settlement, if any, may be uniform.

With vertical loading upon rock or soil whose sustaining power per square foot is equal to or greater than the unit load, the dimensions of the foundation are fixed by the size of the structure, the safe load which can be sustained by the concrete, or by resistance to overturning. If the load is greater than an equivalent area of soil can sustain, the area of the base of the concrete must be enlarged, and the concrete battered or stepped or reinforced. It is a common engineering practise to make the length of the projections or steps of plain concrete one-half the height of the block, and this usually gives good results in buried foundations where the surrounding earth assists to prevent splitting.

The effect of concentrated loading must be considered when designing a footing. (See p. 249.) The pedestal bases for the Boston Elevated Railway were designed, when covering one-half the area of the concrete, with 25% higher unit stresses for the concrete in actual contact than when covering the entire area. Fig. 141, page 476, shows a typical foundation for the columns.*

The following figures are suggested as conservative safe loads, not allowing, on the one hand, for increased strength under concentrated loading, nor, on the other hand, for live loads. These figures are based on ordinary concrete with a factor of safety of 5½ at one month and a factor of 7½ at six months. (See p. 242.)

Safe Loads on Foundations.

Proportions of Concrete by volume.†	Safe Loading	
	Lb. per sq. in.	Tons per sq. ft.
1:1:3	480	35
1:2:4	450	32
1:2½:5	400	29
1:3:6	350	25
1:4:8	280	~~[illegible]9~~ 20

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*George A. Kimball in Journal Association Engineering Societies, June, 1903, p. 351.

†Based on a barrel of packed cement of 3.8 cu. ft., weighing 376 lb. net.

For a vibrating or pounding load these values should be reduced from ½ to ⅓, depending upon the nature of the loading. For a concentrated load the values may be increased according to directions on page 250.

ELEVATION

FIG. 141.—Typical Column Foundations of Boston Elevated Railway. (*See p.* 475.)

[illegible] Taylor & Thompson.

Reinforced Footings. To distribute the load over a large area of soil without carrying the foundations, by successive steps, to a considerable depth and using a large mass of concrete, a single slab may be employed and reinforced with steel to prevent the projection breaking.

Each projection of a footing supporting a vertical load is a beam resisting a uniformly distributed upward pressure whose intensity per unit of area of base is equivalent to the total superimposed load divided by the area of the base. The thickness of the projection and the amount of reinforcing metal, therefore, may be obtained from data in Chapter XIV. The maximum bending moment — and consequently the strongest section of the base — required for most footings is at a point just inside of its line of offset from the wall or column above, but the moment may be calculated

both at this and at some other section, so that the thickness of the concrete and the quantity of steel may be reduced toward the ends of the projection.* The larger part of the metal should of course be placed near the bottom of the base to resist tension.

If two columns are supported by the same base, the concrete between them acts as a cantilever beam loaded at both ends, and must be reinforced near its upper surface, and U-bars or other vertical reinforcement placed as in ordinary beam design.

The reinforcement can be distributed with greatest economy by using rods, of diameters determined by the stresses. A typical reinforced column footing in the Ingalls Building, Cincinnati, O., designed to carry a total load of 812 tons, is shown in Fig. 142, page 478.

Steel I-beams, and in some cases old steel rails, are incased in concrete for column footings. A typical footing, designed by Mr. John S. Branne,† is illustrated in Fig. 143, page 479. In this particular case the situation required a cantilever girder connecting this foundation with the next, but the footing shown is itself designed for a total load of 173 tons, of which 120 tons are dead load and 53 tons live load.

Reinforced concrete retaining walls, whose bases may require similar design to that described for column footing, are discussed on page 490.

Foundation Bolts. It is often difficult to locate bolts in concrete with sufficient exactness for setting a machine. To obviate this difficulty, the head of the bolt should be provided with a large washer‡ to give a good bearing surface, the bolt placed in its approximate position, with washer down, and an iron pipe or a light wooden box placed around the bolt resting upon the washer. When the machine is set, to prevent the bolt from rusting, the iron tube or box should be filled with mortar. In any case the tube or box should be filled with sand before the machine is poured up with sulphur or cement grout, in order to keep these materials from running down the bolt holes.

CONCRETE PILES

Concrete piles may be employed in place of wood where the loading is excessive, and where the durability of timber piles is questioned either

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*For discussion of the distribution of the forces, see Johnson, Bryan, and Turneaure's Modern Frame Structures, 1904, p. 482.

†Journal Association Engineering Societies, February, 1901, p. 142.

‡The washers, which are used for transmitting the pressure of large bolts to the concrete or other foundations, should be carefully designed with heavy ribs so as to transmit a uniform pressure per square inch of area. Neither wrought nor cast iron plates should be used for washers under large bolts.

because of probable worm action or the rotting of the timber. If the bearing is frictional and the piles are driven through ground which is

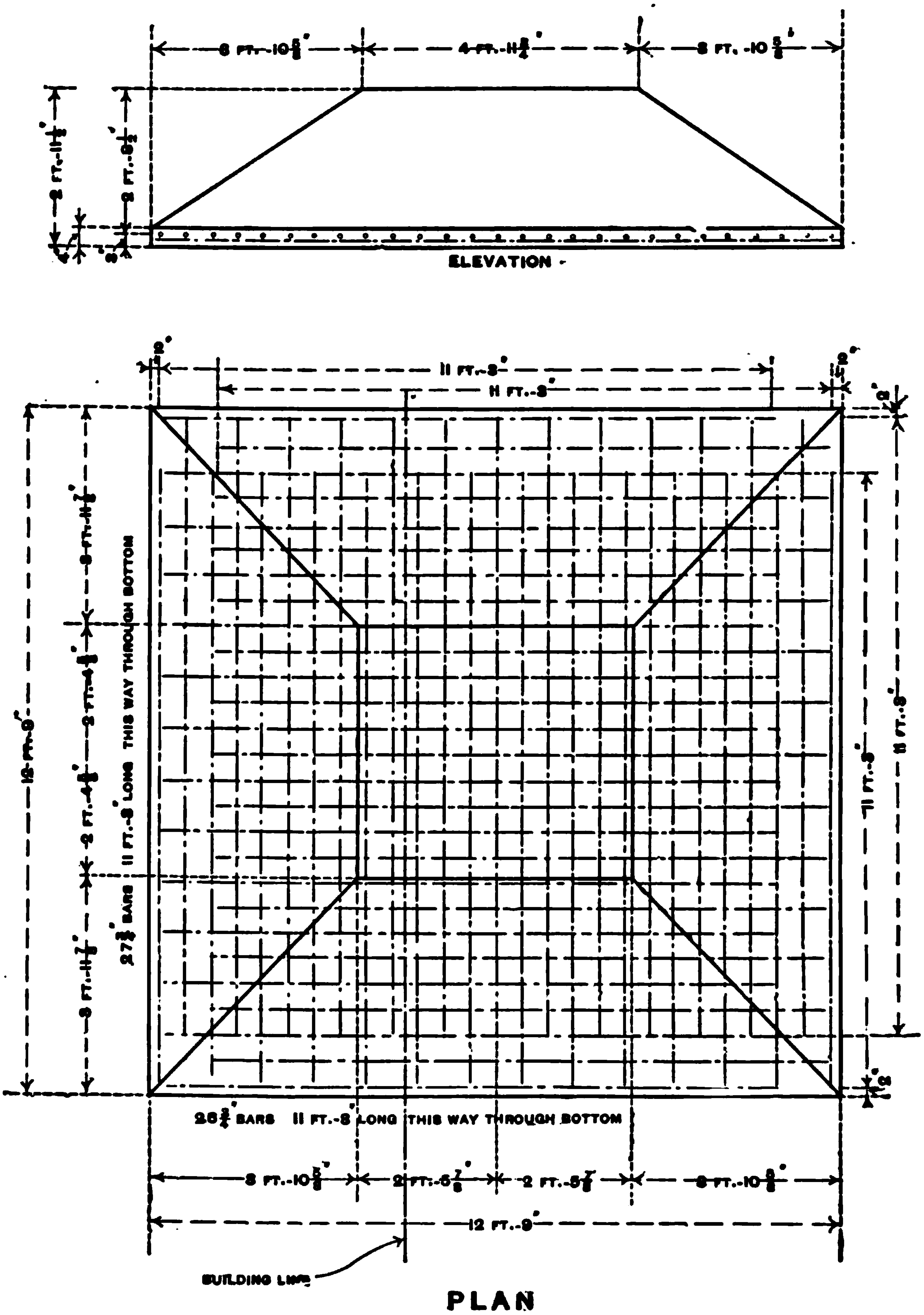

FIG. 142. — Typical Column Footing in Ingalls Building. (*See p.* 477.)

continually wet, there is usually no advantage in concrete over timber piles unless in certain instances where the low level of the ground water or the tide-water is so far beneath the structure that the concrete piles permit the commencement of the foundation at a considerably higher level and thus save excavation and material.

FIG. 143. — Concrete and I-beam Footing. (*See p.* 477.)

Concrete piles are formed (1) in place, or (2) are molded above ground and driven with a pile driver.

Various methods have been suggested for forming the hole into which the concrete is to be placed. One of the patented processes consists in driving a double shell of metal into the ground, removing the inner one, and leaving the outer to form a mold for the concrete. The two shells and pile driver are shown in Fig. 144, page 480. The inner shell or pile core, which is of heavy sheet steel and constructed so that it can be made to collapse for removal from the ground, is placed within the other thinner shell, and driven like an ordinary pile. The core is then collapsed and withdrawn, leaving the outer shell, which is closed at the bottom, to be filled with concrete. By providing considerable taper, additional support is obtained from the soil.

Fig. 144. — Cores for Concrete Piles. (*See p.* 479.)

Another system, illustrated in Fig. 145, consists in driving a single shell with either a concrete or a steel point, then slowly withdrawing it, and filling the space which it occupied with concrete whose surface is kept far enough above the lower end of the tube to maintain the head necessary to resist the pressure of the ground.

In still another method, which is especially adapted for underpinning, the tube is washed down with a water jet to firm strata, and the bottom of the excavation is enlarged by an expanding arrangement to form a base, as shown in Fig. 146.

Piles made in situ may be reinforced if desired.

Reinforced piles which are formed above ground are designed like columns with vertical reinforcement connected at intervals with horizontal wire rods or plates. (See p. 466.) As a circular mold is costly, they are usually made square or triangular in section. They must be driven with a heavy hammer having a short lift. At Brumath, Germany,* for example, the hammer weighed 4½ tons. An elastic capping, such as sawdust enclosed in burlap within a steel jacket, or alternate layers of iron and lead or wood, is necessary when driving. Such piles have been used in England as well as Germany, but they are very costly. The pile shown in Fig. 147, which is similar in principle to the European piles, was designed for the

FIG. 145. — Concrete Piles. (*See p.* 481.)

**Cement*, March, 1903, p. 16.

Hallenbeck Building, New York City, but it was abandoned because of the difficulty in casting and driving.

The designs drawn up in 1903 for the Pennsylvania Railroad Tunnel* under the Hudson River call for a shell of cast iron surrounded by concrete and supported at intervals by steel screw piles filled with concrete.

Fig. 146.—Concrete Pile with Enlarged Footing. (*See p.* 481.)

Sheet Piling. Poling boards of concrete were employed by Mr. Howard A. Carson, Chief Engineer in the construction of the approaches to the East Boston Tunnel. These are described† as follows:

> The excavation was through gravel and clay, and through sand containing some water. Trenches 16 feet long and 16 feet apart were dug to about the level of the bottom of the building foundation. Below the foundation one-half of each trench, or 8 feet in length, was carried down to grade. The bank below the foundation was held in place by means of concrete slabs used as sheet piling, as illustrated in Fig. 148. These slabs were from 6 to 8 feet long, 6 inches wide, and 2 inches thick, and each was reinforced with six square steel rods running the entire length of the slab and shown in Fig. 149. If wooden sheeting had been used, it would have been necessary either to have concreted directly against it and left it in place, or to have pulled the planks as the concrete was filled in. If the first method had been used, the planks would in time have become rotten, leaving a vacant space. If the planks had been pulled, there would have been danger that some of the earth under the building would run and a settlement of the building follow. In order to guard against any slight voids which might have been left in driving, grout was poured in behind the sheeting. This sheeting served not only to hold the bank in place, but was used, in place of a back wall, to waterproof against. The sheeting was not disturbed, and the wall of the Tunnel was built directly against it.

**Engineering News*, Oct. 15, 1904, p. 331.

†Ninth Annual Report, Boston Transit Commission, p. 41.

BRIDGE PIERS

Concrete is employed for bridge piers either as filling for ashlar or cut masonry or for the entire pier. In the latter case, in which the face is also of concrete, the chief question is as to its ability to withstand the wear of the water, the ice, and floating debris. Mr. Martin Murphy* stated as early as 1893 that concrete was generally adopted in Nova Scotia, and with successful results, for abutments and piers "in the most exposed positions, in the midst of strong currents, without any external protection, where exposed to heavy ice floes, to blows from timber rafts, and, in many instances, to undermining by scour." In Nova Scotia it is the common practise to construct the body of the pier of rubble concrete with a 6 to 9-inch facing of richer concrete. In answer to inquiries, Mr. Murphy wrote the authors in 1904: "The concrete piers erected in this Province for the last eighteen or twenty years have withstood the action of the weather, and fulfilled all that was claimed for them in my paper, read before the International Congress in 1893. The erection of such piers and abutments is now in almost universal application in Canada."

In the Kansas City flood of 1903, the piers of solid concrete, although located where they were struck by all the heavy debris which totally destroyed many of the stone masonry structures of the same size, remained practically uninjured.

In 1900 a Committee of the Association of Railway Superintendents of Bridges and Buildings† made the following inquiry: "For what classes of structures do you use Portland ce-

FIG. 147.—Piles designed (but not used) for the foundations of the Hallenbeck Building, New York. (*See p.* 481.)

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*Bridge Substructure and Foundations in Nova Scotia, Transactions American Society of Civil Engineers, Vol. XXIX, p. 620.

†Proceedings of Tenth Annual Convention, Association of Railway Superintendents of Bridges and Buildings, 1900.

ment concrete?" Out of thirty-three replies received, seventeen were in favor of employing this material for both the foundation and neat work of bridges, piers, and abutments.

Plastering of concrete piers and abutments should be prohibited. If a mortar surface is required, an excellent facing, to be placed next to the form as the concrete is laid, is a mixture of one part cement to 2½ parts hard broken stone screenings ½ inch in size and under. Ordinarily, however, no surface finish is required unless superficial treatment is given for the sake of appearance. (See p. 380.)

FIG. 148. — Concrete Sheet Piling in Approaches to East Boston Tunnel. (*See p.* 482.)

Pier Design. Many railroads are substituting concrete for ashlar masonry in bridge piers.

The standard pier of the N. Y. Central R. R., adapted to any height up to 40 feet, is shown in Fig. 150, page 486.* The width, which depends upon the length of span, is as follows:

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*Arranged from original drawing, for which the authors are indebted to Mr. Wilgus.

Spans up to 40 feet width, A, = 4 ft. 0 in.
Spans 40 to 60 feet width, A, = 4 ft. 6 in.
Spans 60 to 80 feet width, A, = 5 ft. 0 in.
Spans 80 to 100 feet width, A, = 5 ft. 6 in.
Spans 100 to 125 feet width, A, = 6 ft. 0 in.
Spans 125 to 150 feet width, A, = 6 ft. 6 in.
Spans 150 to 200 feet width, A, = 7 ft. 0 in.
Spans 200 to 250 feet width, A, = 7 ft. 6 in.
For skew crossings, increase width, A, if necessary.

Foundation is varied to suit local conditions. Concrete 1:3:6 is employed for it unless stone masonry is cheaper. The starkweather is carried 2 feet above high water, and its cap is of 1:1:2 concrete. The coping of the pier is reinforced with galvanized wire netting or wire cloth, a somewhat unusual requirement.

The Illinois Central R. R., in their 1904 design, reinforce the surface of piers with vertical and horizontal steel rods, and imbed a single I-beam in the pointed nose at each end of the pier.*

The Chicago, Milwaukee & St. Paul Railway Company takes the extra precaution to strengthen the noses or starlings of its concrete piers only at points where there is considerable ice and driftwood.† They build a 7-inch street car rail into the nose of the pier, with the head projecting slightly from the concrete. Other roads also show no reinforcement in their standard design.

It would appear that reinforcement is probably unnecessary except in situations where the piers are subjected to unusual impact.

All of the roads named above have piers in streams which subject them to considerable wear from ice and drift, and the concrete has proved satisfactory.

FIG. 149.—Sheet Piling. (*See p.* 482.)

FOUNDATIONS UNDER WATER

The best and most durable concrete foundations, especially in work in sea water, are laid within cofferdams from which the water has been pumped, or in pneumatic caissons. However, because of the

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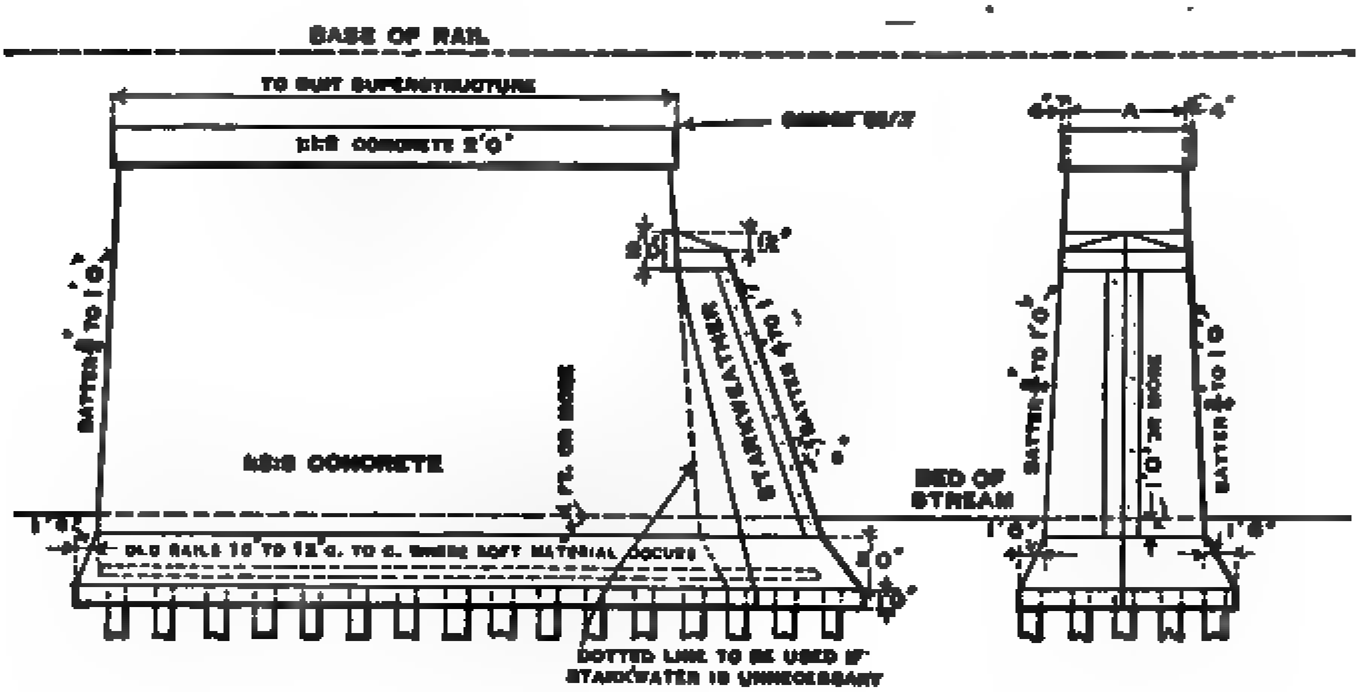
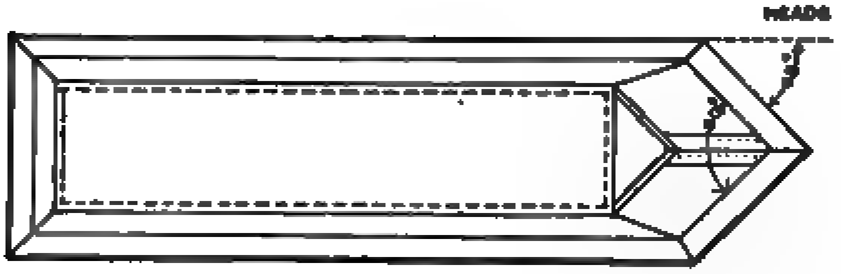
*From drawing kindly furnished by H. W. Parkhurst, Engineer.
†Authority of C. F. Loweth, Engineer.

difficulty and expense of these methods, they cannot usually be followed. If the bottom is prepared by dredging, and, if necessary, driving piles, good practise permits the use of a single line of sheeting, suitably supported with rangers, to prevent the wash of the water and keep the concrete from spreading.* Permanent metal cylinders are sometimes sunk in place of the sheeting.

Methods of laying concrete under water are described in Chapter XVII, page 392, and the effect of sea water upon concrete is discussed by Mr. R. Feret in Chapter XVIII.



SIDE AND UP-STREAM ELEVATIONS



PLAN

FIG. 150.— Standard Concrete Bridge Pier, N. Y. C. & H. R. R. R., W. J. Wilgus, Chief Engineer. (*See p.* 484.)

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*See Foundations for New Cambridge Bridge, Boston, by Sanford E. Thompson, *Engineering News*, Oct. 17, 1901, p. 283.

For under-water work, a larger factor of safety should be employed than for work above ground, the concrete should be slightly richer in carefully selected cement, and the aggregate so proportioned as to give a dense and impervious mixture.

Concrete for the foundations of walls and piers for high office buildings is usually laid in oblong or circular caissons of steel or wood,* after excavating under air pressure. Steel pipes are sometimes sunk with the aid of the water jet, and afterwards filled with concrete.†

***Engineering News*, Sept. 26, 1901, p. 222.

†Jules Breuchaud, Transactions American Society of Civil Engineers, Vol. XXXVII, p. 31.

CHAPTER XXV

DAMS AND RETAINING WALLS

For walls to resist the pressure of earth or water, concrete frequently possesses marked advantages over other classes of masonry. With proper management, in many localities its cost may be brought below that of rubble masonry. Its adaptability for thin walls and for certain classes of face work often make it a suitable substitute in complicated designs for first-class masonry, with a consequent large saving in cost. In combination with steel its possibilities for special designs are almost unlimited, and the future will see marvelous advances in its use for ordinary engineering and hydraulic construction.

Water-tightness, often an essential element for this class of structures, has received general treatment in Chapter XX, page 416. Portland cement concrete may be made water-tight more readily than stone masonry laid in mortar of similar proportions to the cement and sand in the concrete, since large voids or stone pockets in the concrete are more easily prevented than the "rat-holes" so frequently found in the bedding of stones in mortar. Moreover, skill in laying combined with special treatment of the surface or the addition of certain ingredients permits construction in concrete — strengthened with steel reinforcement — of thinner walls for resisting the flow of water than is possible in stone masonry.

DESIGN OF RETAINING WALLS

Retaining walls to support the pressure of earth may be designed

(1) of gravity section with plain concrete;

(2) of thin section, reinforced and supported by buttresses or counterforts;

(3) of thin reinforced section with spreading base or footing.

Another plan, not included in this classification, which is sometimes adapted to cellar wall construction (see p. 461), consists in imbedding the base and supporting the top of the wall with timber or steel, so that the concrete forms a vertical slab supported at top and bottom.

An economic advantage of concrete over stone masonry lies in its adaptation to thin sections supported by counterforts or a spreading base, or a combination of these.

Buttresses in either concrete or stone masonry are likely to occupy valuable space in front of a wall and to present an unsightly appearance. Counterforts, that is, projections or buttresses running back into the filling, are of comparatively little advantage in stone masonry because, under pressure, the wall is liable to break away from them, and it is more economical to place all the material in the wall itself. With concrete, however, different conditions obtain, as the counterforts, combined with aprons or spreading footings weighted with earth, can be securely tied to the longitudinal wall by reinforcing metal. For comparatively low walls up to, say, 10 feet in height, a straight reinforced wall with a spreading footing is satisfactory; it, in fact, becomes a beam or slab supported at the bottom. The principles of design are discussed in succeeding pages.

Whatever the type of design, the securing of a firm foundation is essential. Piles may be necessary, or, to avoid danger of sliding, an inclined or stepped base may be required.

The pressure exerted by earth varies largely with the character of the soil and the amount of water which it contains, and numerous theories have been advanced for calculating it, or for treating it graphically. The principles of design suggested in the following pages may be adapted to these more elaborate theories, if the size of the structure warrants the investigation. The data presented may be applied with safety under all ordinary conditions.

Gravity Section. The thickness of base of a retaining wall of gravity section, that is, one in which the earth pressure is resisted by the weight of the masonry, is generally taken without mathematical calculation as a certain ratio of the height of the wall. An easily remembered rule is to make the base ⅜ of the height. The table of empirical values adopted by Mr. Trautwine* for thickness of base of wall to resist earth pressure under average conditions is in accordance with good engineering practice. While he gives no values for concrete, they may safely be assumed equivalent to those for cut stone laid in mortar, which are as follows·

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*Trautwine's Engineer's Pocket-Book, 1902, p. 606.

Thickness of Retaining Walls of Gravity Section.

By John C. Trautwine. (*See p.* 489.)

Ratio of Height of Earth to Height of Wall.	Thickness of Base as ratio to Height of Wall.	Ratio of Height of Earth to Height of Wall.	Thickness of Base as ratio to Height of Wall.
1.	0.35	2.	0.58
1.1	0.42	2.5	0.60
1.2	0.46	3.	0.62
1.3	0.49	4.	0.63
1.4	0.51	6.	0.64
1.5	0.52	9.	0.65
1.6	0.54	14.	0.66
1.7	0.55	25.	
1.8	0.56	or more	0.68

The height of the wall is assumed to be measured above the surface of the ground in front of it.

The batter of the face of a retaining wall is customarily limited to 1½ inches to the foot, and the back is usually vertical. This fixes the width on top.

The values in the table may be employed for long walls of concrete with no reinforcement. In many cases, because of the monolithic character of concrete, a ratio of thickness to height from 10% to 20% less may be adopted with safety, if the character of the filling back of the wall precludes excessive pressure, and if the base is slightly spread. For more accurate determinations of gravity sections, the principles which follow relating to reinforced designs are applicable.

Reinforced Wall with Spreading Base. For a reinforced retaining wall a simple type is an inverted T-section. A 5-foot and a 10-foot wall of this style, designed by Mr. A. L. Johnson,* are illustrated in Figs. 151 and 152, page 491. These walls have sufficient metal reinforcement, according to the law stated on page 378, to withstand temperature changes without contraction joints.

In designing a retaining wall of inverted T-section, the upright slab is calculated as a beam supported at its lower end, with the earth pressing against it, and the thickness and reinforcement must be sufficient to withstand the bending moment and the shear due to this pressure. The thicknesses and reinforcements of the projections at the base are determined by combining the forces of pressure and weight, as outlined below.

Reinforced Wall with Counterforts. The general principles of design of a wall strengthened by counterforts is similar to that just described, but

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* Especially for this Treatise.

the stresses are differently distributed. In the wall in Fig 153, also designed by Mr. Johnson, the counterforts, which are in tension, are held at their base by the apron and the weight of earth upon it. They therefore sustain the entire horizontal pressure of the earth, which is confined by the face of the wall acting as a slab supported at four edges, since the top coping acts as a reinforced beam. The face of the wall in this case is reinforced with metal upon its outer face, since the pressure tends to bulge it. The steel is designed with so large a section that expansion joints are not needed.

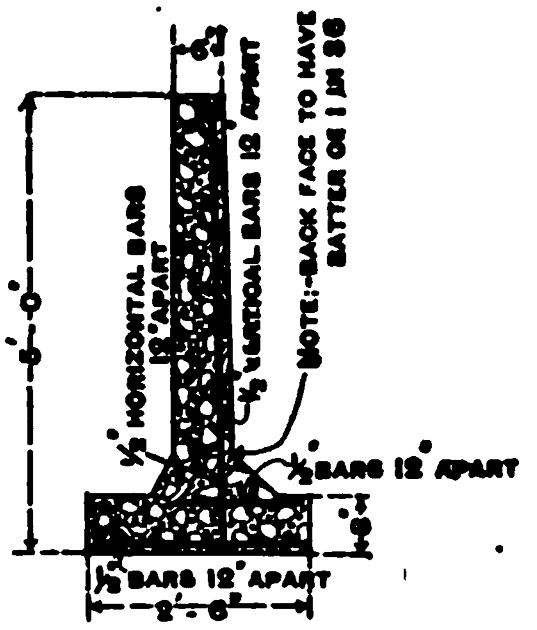

FIG. 151. — Five-foot Reinforced Retaining Wall. (*See p.* 490.)

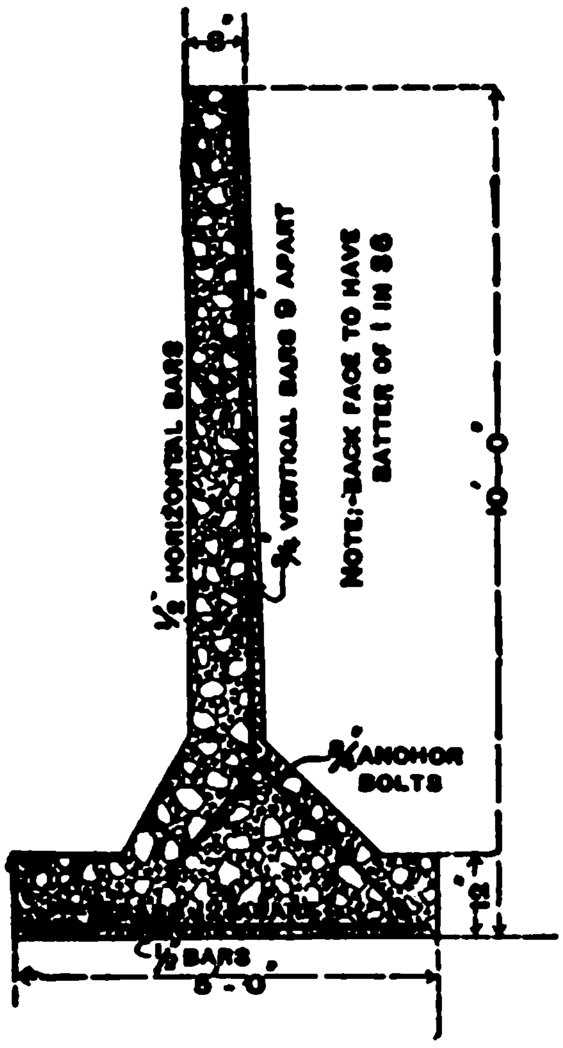

FIG. 152. — Ten-foot Reinforced Retaining Wall. (*See p.* 490.)

Earth Pressure. In a wall of gravity section, the chief requirement is to provide masonry of such thickness and weight that the wall cannot be overturned. Therefore, in practise, empirical rules and tables, such as are given above, may be safely employed.

In reinforced walls of thin section, on the other hand, the moments and shears must be calculated and the forces of weight and pressure must be determined as accurately as is possible, considering the variable character of different soils.

Earth pressure is *active* or *passive.* In a trench, it is evidently much easier to prevent the earth on either side from falling in than it is to force

back the earth by pressing against it. Similarly, if a wall is built in the trench or if earth is filled against it, the active pressure tending to overturn it is many times less than the passive pressure on the opposite side which tends to prevent overturning. The center of pressure in both cases is at points two-thirds the distances from the earth surfaces to the bottom of the wall. As the passive pressure of the earth in front of a wall only acts at the point of overturning, it is usually disregarded and the earth in front merely assumed to balance an equivalent depth of earth back of it.

If the back of the wall is vertical,—a condition apt to be nearly true of a reinforced concrete wall, — the direction of the resultant pressure, before allowing for the friction against the wall, can be proved to be parallel to the surface of the earth, whether the surface is level or inclined. The friction between the earth and the back of the wall, which is usually assumed as tending to prevent overturning, is equal to the pressure against the wall multiplied by the coefficient of friction, and acts downward along the direction of the wall. This force may be combined with the direct pressure.

FIG. 153. — Retaining Wall with Counterforts and Apron (*See p.* 491.)

The following table of pressures determined by Prof. Mohr's graphical method, the results from which agree with Rankine's formulas, gives horizontal earth pressures per square foot, disregarding friction, for different heights of wall, based on an angle of repose of earth of 45° — a fair assumption under average conditions — and also pressures in which this horizontal pressure and the vertical friction (whose coefficient in this case is $\tan 45° = 1$) are combined. This resultant force acts downward at an angle of 45° with the vertical back of the wall. If an angle of repose of 30° with the horizontal is assumed, the active horizontal pressures are nearly double, and the passive horizontal pressures nearly half. For other

heights of wall, the horizontal pressures with the same angle of repose are directly proportional to the heights.

Average Unit Earth Pressure upon Vertical Walls of Different Heights. (*See p.* 492.)

Active			Passive		
Depth of Earth feet	Average Horizontal Pressure lb. per sq. ft.	Average Pressure Acting at 45° * lb. per sq. ft.	Depth of Earth feet	Average Horizontal Pressure lb. per sq. ft.	Average Pressure Acting at 45° * lb. per sq. ft.
5	94	133	1	640	905
10	188	266	2	1280	1810
15	282	399	3	1920	2720
20	376	532	4	2565	3630
25	470	665	5	3210	4540
30	564	797	6	3850	5450
35	658	931	7	4490	6350
40	752	1062	8	5130	7260

This table assumes (1) horizontal surface of earth, (2) vertical back on wall, (3) weight of earth per cubic foot, 110 pounds (4) angle of repose, 45°.

Weight of Earth. In the calculation of retaining walls, the weight of earth in place is a prime factor. The weights of dry material, based upon experiments by the authors, are represented in the following table. Most of the figures for weights of earth give the weights per cubic foot after excavation in a loose or a compacted condition. In the authors' experiments the excavation was measured, so that the weights represent the material in place. As fills will eventually assume much the same characteristics as earth in original excavation, the figures may be employed for either natural earth or filled material. The weight of earth containing water varies with the character of the material and with the conditions. Gravel containing ordinary moisture weighs about 2% more than dry gravel, and sand may weigh from 3% to 10% more, depending upon its fineness, since fine sands absorb the most water. Wet muck weighs about 75 lb. per cubic foot. These percentages assume that the bank is provided with natural drainage; if the earth is literally filled with water which cannot run off, its weight will be increased by a quantity of water nearly equal in volume to the voids in the material, which vary with the character of the material from 15% to 50% of the bulk of the earth in the bank.

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*Includes effect of friction.

Many of the values appear high, but they are the result of careful tests.

Average Weight of Ordinary Earth before Excavation.

	Pounds per cu. ft.
Sand	105
Gravel	135
Gravelly clay	130
Loam	90
Hard pan	130
Dry muck	40

Copings. A coping may be formed on a concrete retaining wall which will shed water, and look nearly as well as cut stone. A mortar surface may be obtained next to the forms by plastering the inside of the form itself with mortar just in advance of placing the concrete. If the top is sloped toward the back of the wall and troweled with a plasterer's trowel so as to flush the cement to the surface, the water will run off. Forms for a coping to be built in 12-foot lengths are illustrated in Fig. 154.* Using 2-inch plank, the frames should be placed 4 or 5 feet apart. Besides the horizontal triangular beadings or moldings shown, vertical moldings are placed in a coping 12 to 16 inches thick, at intervals of about 12 feet.

Fig. 154.— Forms for Concrete Coping. (*See p.* 494.)

DAMS

Concrete is a suitable substitute for stone masonry (*a*) in gravity dams, where the masonry is laid in large masses, whenever the cost per cubic yard of concrete rubble is cheaper than stone masonry of equal quality, and (*b*) in curtain or arch dams of thin section reinforced with steel.

Concrete of cement, sand, and crushed stone cannot always compete in price with rubble masonry laid in cement mortar, because, although the labor cost of laying concrete is less, more cement is required per cubic yard; but by introducing large stones into the concrete, the percentage of cement per cubic yard may be reduced to the same quantity or even less than in water-tight rubble masonry. Therefore, the concrete rubble is apt to be the cheaper, since the cost of crushing the stone for the concrete

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**Engineering News*, July 9, 1903, p. 37.

is small compared with the difference in expense of employing skilled masons or unskilled labor.

Methods of laying rubble concrete and the calculation of the quantity of cement per cubic yard are discussed in Chapter XVII, pages 391 and 389. As is there stated, the concrete must be of soft, mushy consistency so that the large stone may be properly imbedded.

The relative cost of rubble concrete and stone masonry depends upon the price of cement at the work and local conditions. The dam at Boonton, N. J., a section of which is shown in Fig. 155, p. 496, contains 240,000 cubic yards of concrete rubble, and was built at a contract price, not including the cement, of $1.98 per cubic yard. Only 0.6 barrels Portland cement were used per cubic yard, although the proportions of the concrete matrix were 1 : 2¾ : 6½. This small quantity of cement was due to the large proportion of stones which averaged from one yard to 2½ yards each and occupied 55% of the total volume. The contract price mentioned includes the preparation of the large stones and the crushed stone, and their transportation from a quarry three miles away. It is believed by the authors that the price and also the quantity of cement per cubic yard represent minimum figures in first-class construction, but the force account showed that the contractor was making a fair profit, and inspection of the work and its water-tightness prove that there was no skimping in the use of cement. On this particular job the quotation of the highest bidder was nearly double the accepted price.

With reinforced concrete the engineer is able to branch out into special types whose design may be applicable to local conditions.

Design of Gravity Dams. A foundation must be secured which will resist the pressure upon it and prevent percolation of water under the masonry. The end connections with the adjacent soil or rock must also be carefully considered. The section of the dam must be of such thickness and design as to prevent (1) leakage, (2) overturning, and (3) sliding.

Leakage through a concrete dam of gravity section need only be considered to the extent that no careless work be allowed.

To resist overturning, it is customary to require that the resultant of all the forces of pressure and weight shall pass through the middle third of the base. Dangerous sliding need not usually be feared if the dam is designed to resist overturning. In considering the resistance of friction, Mr. Joseph P. Frizell* states that smooth stone slides on smooth stone

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*Frizell's Water Power, p. 19.

under a horizontal force of two-thirds its weight, and to slide on gravel or clay, stone requires a force nearly equal to its weight.

The pressure of the water upon any submerged surface is equal to the area of the surface in square feet times the weight of a cubic foot of water times the depth of the center of gravity of the surface below the water level. This pressure tends to overturn the dam, and is resisted by the weight of the dam, and in some cases, where the up-stream face slopes, by the weight of the water upon the dam.

The treatment in Frizell's Water Power of the location of the center of pressure, and the moment produced by it, is especially clear and practical.

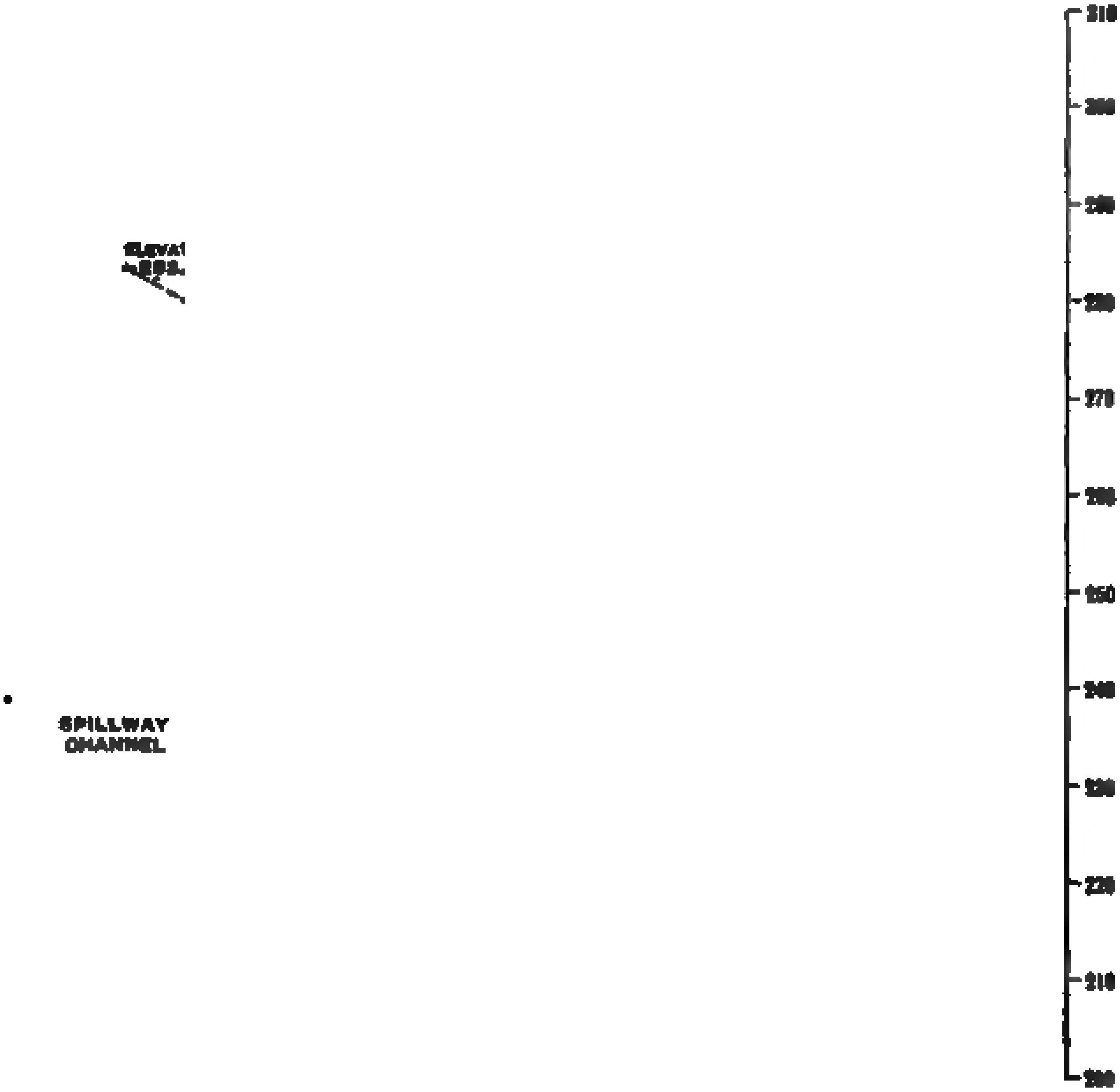

FIG. 155.—Section through Overflow of Boonton, N. J., Dam. (*See p.* 496.)

Fig. 155 represents a section through the overflow of the concrete dam at Boonton, N. J., the construction of which is described on page 391.

The extreme height of the dam at the highest point above the foundations is 110 feet. An interesting practical test of the water-tightness of concrete occurred when the reservoir was filled. A vertical well was left in the dam in order to provide access to two drainage gates, and although the water in the reservoir is 100 feet deep, and is separated from the well by only 5 feet 6 inches of concrete mixed in the proportions 1 : 2¾ : 6¼, the well remains entirely dry.

Reinforced Dams. The aim in reinforced dams is to reduce the quantity and cost of materials, and at the same time to permit a much broader base, and a sloping water-tight deck for the up-stream face. The water pressure is thus made to increase instead of oppose stability.

A section of such a dam at Schuylerville, N. Y., 250 feet long and 25 feet high, is shown in Fig. 156. The buttresses are on 10-foot centers, and support a deck tapering from 8 inches to 12 inches thick, while the overfall apron is 8 inches thick. A foot-bridge lighted by electric lights passes through under the crest, giving access from the mill to the railway platform on the other bank.

FIG. 156. — Section of Reinforced Concrete Dam at Schuylerville, N. Y. (*See p. 497.*)

Arched Dams. Curved dams, designed in plan as a single arch, convex up-stream, are considered by foremost authorities to be of doubtful economy, as the extra length requires more material than is saved by the reduced cross-section.

Recently, a type of dams consisting of a series of arches supported by piers or steel lattice work has been suggested, and this idea may receive further development through the introduction of reinforced concrete.

A dam in the form of a buttressed wall with a vertical up-stream surface has been suggested by Mr. George L. Dillman,* the dam in plan consisting of parabolic arches.

The design for a dam at Ogden, Utah,† consists of a number of piers, triangular in vertical section, forming buttresses to support an up-stream sloping face composed of circular concrete arches from 6 to 8 feet thick. The arches are designed to be covered on their upper surface with ¼-inch steel facing. The top of the dam, which is also formed by arches between the piers, carries a roadway.

CORE WALLS

Concrete is largely superseding rubble masonry for core walls in earth dams and dikes. The forms can be roughly made without reference to the appearance of the faces, while a thin wall of concrete may be built water-tight more easily than one of rubble masonry. Unless reinforced, core walls are generally of the same thickness as those of rubble masonry. The Natural cement concrete core wall of the Sudbury Dam, built by the Boston Water Commissioner and his successor upon the work, the Metropolitan Water Board of Massachusetts, is 2 feet thick at the top, with a batter of one in fifteen on both faces, until it reaches a maximum width of 10 feet. At Spot Pond Reservoir, several dikes with core walls of Portland cement concrete, of 15 to 18 feet average height, are 2½ feet in thickness throughout.

The dike for the Jersey City Water Supply Company at Boonton, N. J., is designed for a total height of 54 feet. The lower 30 feet is 4 feet 8 inches thick, and at this height it begins to batter, so as to reach a width of 3 feet at the top.

Although core walls may often be economically built of rubble concrete, the stones must be of smaller size, and cannot occupy so large a volume of the mass as in gravity dams, since the sections are thinner. In the construction of the Boonton Dike, mentioned above, one contractor was placing rubble to the extent of 20% of the total mass, while another was placing 33%. In the former case the stones were loaded on to derrick skips and unloaded by hand; in the latter case, they were hooked by the derrick. This 33% probably represents a maximum for a wall 5 feet thick or less.

Since a thin wall of reinforced concrete may be made equally strong, and more elastic than a thick wall of plain concrete, reinforcement may eventually be employed to reduce the section, and therefore the quantity of material.

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*Transactions American Society of Civil Engineers, Vol. XLIX, p. 94.

†Henry Goldmark in Transactions American Society of Civil Engineers, Vol. XXXVIII, p. 290

CHAPTER XXVI

ARCHES, TUNNELS, AND CONDUITS

Since the principal stresses in arches are compressive, concrete is peculiarly suitable for all classes of arched structures. Eccentric loading may be provided for by increasing the thickness of the concrete at the points of greatest stress, by steel reinforcement, or by both. The steel may also prevent failure of thin sections of the arch from excessive stresses due to suddenly applied loads or to settlement of the foundation.

Concrete is supplanting cut stone in arch bridges because of its relative cheapness. Although not entirely acceptable from an architectural standpoint because of the difficulty in obtaining a satisfactory surfacing, several methods of treating the face have been used with fair success. (See p. 380.) This objection may also be met by facing the arch with cut stone. Methods of arch design are treated on page 514.

Concrete arches and conduits are likely to be cheaper than brick even at the same price per cubic yard, because the greater strength of the concrete makes a thinner section possible.

Tunnels (see p. 509) and subways (see p. 512) are now built almost exclusively of concrete, or of combinations of concrete and steel.

CONDUITS

Sewer and water conduits of almost any size or shape may be built of concrete. In the larger sizes, and in conduits under pressure, steel reinforcement occasionally may be advisable from the standpoint of safety and economy.

Concrete was first used in conduits to form in bad ground a foundation for a brick invert. Later it was adopted instead of brick for the entire arch, and finally, in many instances, the brick invert lining has also been replaced by concrete.

While concrete may not be preferable to brick in all localities and under all conditions, its advantages are sufficient to always warrant a very careful investigation of its adaptability to the work in question.

As far back as 1850 sewers and aqueducts of béton or béton-coignet (see p. 1) 8 feet in diameter were constructed in France. The materials consisted of $\frac{1}{4}$ part heavy Paris cement, one part hydraulic lime, and 5

parts sand.* Some of these structures, notably the viaduct of La Vanne, are said to have cracked and flaked.† Not until the beginning of this century, however, was concrete extensively used for conduit construction, although in the extreme western part of the United States for a number of years it had been employed to a certain extent upon irrigation works for lining both canals and tunnels, a thickness of 4 or 6 inches corresponding to 8 inches or two rings of brickwork.‡

Comparison of Brick and Concrete Conduits. Even with no reinforcement Portland cement concrete is unquestionably stronger, when properly proportioned and laid, than brickwork of equal thickness. Therefore, even if the cost per cubic yard of the two materials, including centering, is practically the same, the concrete is made more economical than brick by the adoption of a thinner ring, or a ring of varying thickness proportioned to suit the actual stresses.

A comparison of data shows that concrete conduits can be built at one-fifth to one-third less cost than brick conduits of equal diameter. Williamsport,§ Pennsylvania, furnishes an example where bids were obtained for brick, plain concrete, and reinforced concrete. The contract bids on the plain concrete section averaged considerably less than the brick, and the bids on reinforced construction the lowest of the three.

Referring to the reconstruction of sewers necessitated by the New York Subway, Mr. William Barclay Parsons, Chief Engineer, makes the following statement in his report to the Board of Rapid Transit Commissioners: ||

> During the year 1901 an experiment was made to construct sewers *in situ* in concrete. The first experiment gave such satisfactory results that the principle has been extended to other sewers in a similar manner during the year, except that instead of building the arch of brick, as was done at first, the whole sewer in many cases has been built of concrete. The advantages of this form of construction are that a perfectly smooth surface is obtained without joints, with all connections, curves, cut-waters and other details molded to perfect lines, and that construction can be carried on more rapidly.

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*Leonard F. Beckwith in Transactions American Society of Civil Engineers, Vol. I, p. 108. Mr. Beckwith also gives a table of strength of béton from Michelot.

†O. Chanute in Transactions American Society of Civil Engineers, Vol. X, p. 307.

‡William Barclay Parsons in Transactions American Society of Civil Engineers, Vol. XXXI, p. 314. See also description of the lining of a water works tunnel with concrete in Massachusetts, by Desmond Fitzgerald, Transactions American Society of Civil Engineers, Vol. XXXI, p. 394. See also References, Chapter XXIX.

§*Engineering News* Supplement, Sept. 11, 1902, p. 92.

||Report for 1902, p. 271.

It is reported that these concrete sewers have cost one-third less than brick sewers of the same size.*

Concrete, especially if reinforced, has another great advantage over brick, in that it is able to withstand internal water pressure.

Water-Tightness of Conduits. Water-tightness is to a certain extent dependent upon the proportion of cement to sand. If for a concrete conduit the sand and cement are mixed in the same proportions employed for the mortar between the joints in a brick sewer, the structures ought to be equally impervious. For example — a 1 : 2½ : 5 concrete should be as water-tight as brick laid in 1 : 2½ mortar.

If the concrete invert is laid in separate sections, these may be connected by a stepped joint similar to one of the many joints between the different courses in brickwork.

The best proof, however, of the practicability of laying concrete conduits which will prevent the percolation of water, is the fact that sections 4 inches and 6 inches in thickness, which satisfactorily withstand water pressure, have been and are still being built.†

Lime thoroughly hydrated or slaked, or Puzzolan cement, may eventually prove to be the most satisfactory ingredient to mix with Portland cement concrete as a substitute for a portion of the cement, its extreme fineness assisting in filling the minute voids and thus increasing the imperviousness.

The general subject of water-tightness is discussed in Chapter XX.

Durability of Concrete Inverts. Concrete inverts have proved in practise to be equal, if not superior, in durability to the best hard-burned brick.

The hardness and smoothness of surface obtainable with concrete reduce the friction to a minimum and render it less liable to erosion than are other materials. Concrete sewers built at Duluth, Minnesota, furnish a practical example of the ability of Portland cement mortar to resist erosion. After twenty years of wear, they show no appreciable deterioration or enlargement in diameter, while brick sewers laid at the same time required rebuilding after six or seven years. A section of the Duluth drains, about 2 000 feet long and 4 feet in diameter, was built on a 13 per cent. grade where the velocity of the water was 42 feet per second, with an invert of flat granite flags laid with 1 : 1 Portland cement joints. The flow of water during heavy storms was tremendous, carrying down with it quantities of sand and boulders, but after two years of wear the invert

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***Engineering News*, March 6, 1902, p. 201.

†See Sewers and Conduits in References, Chapter XXIX.

showed ridges of mortar between the granite flags, indicating that the Portland cement mortar was more durable than the granite.

Experiments by Mr. Eliot C. Clarke indicate that Portland cement mortar in proportions 1:2 will withstand erosion better than either richer or leaner mortar. (See p. 125.)

Design of Concrete Conduits. The selection of shapes and sizes of conduits suitable for different flows of water and sewage is treated in literature on hydraulics and sewerage. If the material adopted is concrete, it should be of a minimum thickness consistent with good workmanship, strength, and durability. Steel reinforcement reduces the quantity of concrete required for the larger sizes, but for a diameter of 3 feet or less there is no practical advantage in its use unless the conduit is under pressure, because the minimum thicknesses which can be advantageously placed in a sewer trench are sufficient to withstand all strains. Even in larger conduits the use of steel reinforcement is not usually advisable under ordinary conditions, because of the cost and the difficulty of properly placing the metal.

In preference to an entire concrete section, many engineers advocate an invert of one or sometimes two rings of brick laid in a concrete foundation and surmounted with an arch of either brick or concrete. Others favor a concrete invert paved with a granolithic wearing surface, — thoroughly troweled, — from one-half to one inch thick.

The design of a conduit is dependent upon the depth and character of the material through which it passes, but a few typical illustrations may afford hints for special cases. The proportions of the concrete should be carefully determined by an examination of the aggregate at hand. (See Chapter XI, page 183.) A mixture of one part packed cement, 2 parts sand, 4 parts stone or gravel, is rich enough for important work, while proportions as lean as 1:4:8 may sometimes be employed for sub-foundations or backing. In most cases the selection will lie between these two extremes. Natural cement, because cheaper than Portland, is especially adapted for foundations and filling which are not subject to stress or to wear. Puzzolan cement is also suitable in many instances.

The Weston Aqueduct of the Metropolitan Water Works, Massachusetts, built on a gradient of one in 5 000, has in loose earth a typical section shown in Fig. 157. In compact earth the excavation is narrower, and the width of base is reduced as shown by one or the other of the dotted lines, AB or CB. In embankment, the foundation is carried lower and horizontal reinforcing rods are sometimes placed at intervals just below the brick invert lining.

In the Chicago Clearing Yards[*] drainage is accomplished by concrete sewers. The 36-inch and 42-inch diameter mains are 8 inches thick, the 48-inch diameter are 10 inches thick, and the 84 and 90-inch mains, 12 inches thick. The ring in each size is of uniform thickness, and the lower portions of the interior surface are covered with a ½-inch coat of plaster.

FIG. 157.—Typical Section of Weston Aqueduct in Loose Earth. (*See p.* 502.)

In large concrete conduits, even when of circular shape, and passing through material which needs no foundation, it is good practice, whether or not reinforcement is employed, to thicken the walls at the spring of the arch. At Williamsport, Pennsylvania, a 11-foot concrete sewer, suggested as a possible substitute for a 4-ringed brick sewer, was designed 13 inches thick at the crown and invert, and 19½ inches thick at the haunches with no reinforcement.

The Jersey City Water Supply Company constructed in 1903 a conduit reinforced with twisted steel. A typical section, taken through a manhole, is shown in Fig. 158, as designed by Mr. William B. Fuller. Longitudinal reinforcement consists of $\frac{7}{16}$-inch rods spaced about 18 inches apart, and circumferential reinforcement is formed by rings of ⅜-inch rods about 12 inches apart. Through rock open cut the metal was placed only in the

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[*] See article by E. J. McCaustland, *Cement*, Sept., 1902, p. 265.

arch, and as far down on each side as the filling would extend. The open-cut conduit is shown in process of construction in Fig. 110, page 370.

At Kalamazoo, Michigan, Mr. George S. Pierson adopted for a creek culvert* a section shown in Fig. 159.

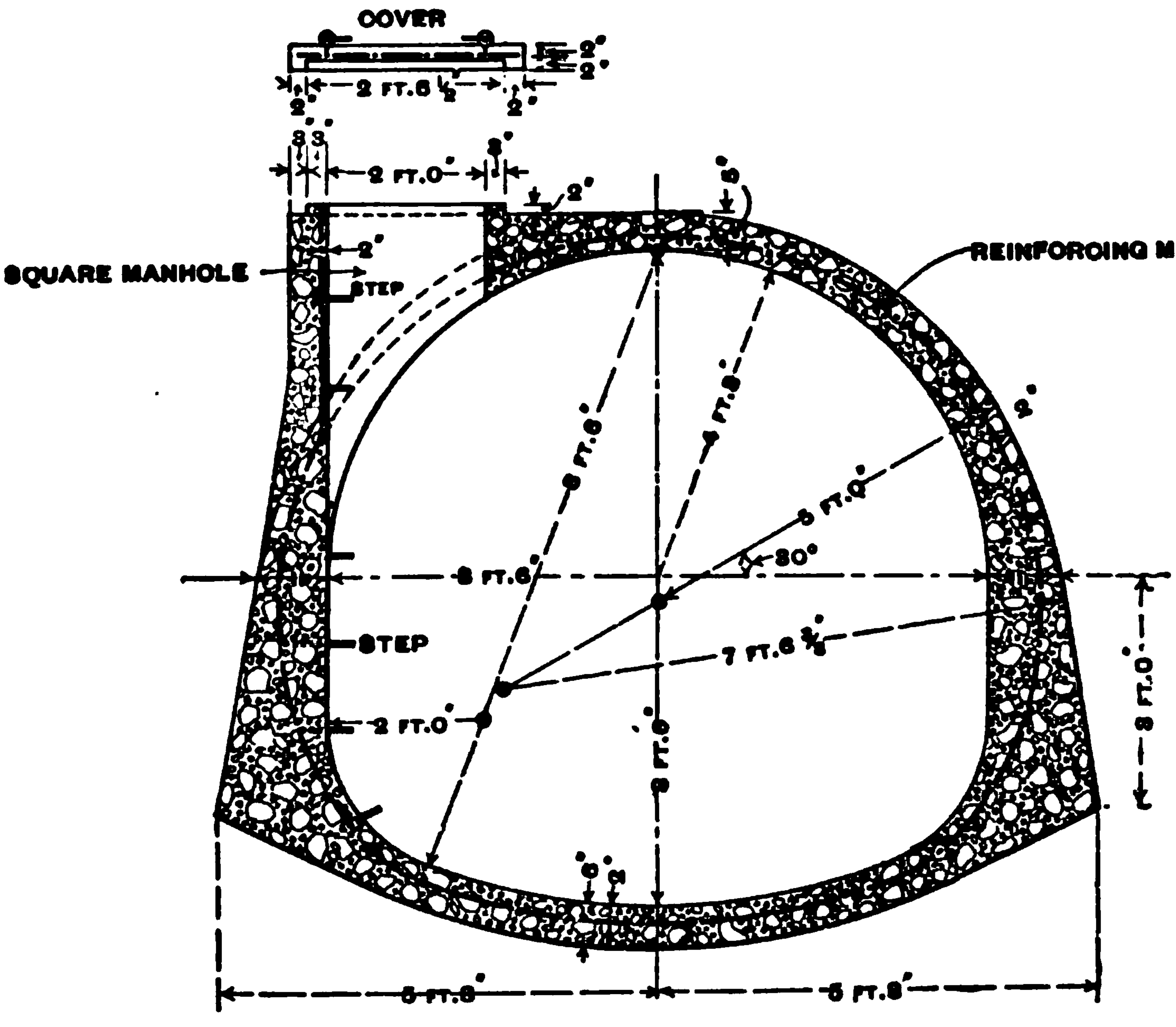

FIG. 158. — Typical Section of Jersey City Water Supply Conduit in Loose Earth. (*See p.* 503.)

At Grenoble, France,† in 1902, a concrete-steel penstock was built to withstand a pressure of 65 feet head of water. The thickness of wall is from 8 to 10 inches, reinforced with longitudinal bars ¼ to ½ inch diameter and circular hoops ⅜ to ⅞ inch diameter, forming a mesh about 4 inches square.

Thickness of Conduits. Mr. Fuller's general rule‡ for determining the thickness of concrete in conduits is as follows:

If concrete is not reinforced and ground is good, — able to stand without sheeting, — make crown thickness a minimum of 3 inches, and then one

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*Described in *Engineering News*, Feb. 12, 1903, p. 163.

†*Engineering Record*, Mar. 7, 1903, p. 249.

‡Personal correspondence.

inch thicker than diameter of sewer in feet. Make thickness of invert same as crown except never less than 6 inches. Make thickness at haunches one and a half times thickness of crown, but never less than 6 inches. This rule is expressed in the following table:

Thickness of Conduits.

Diameter of Conduit.	Thickness of Crown, inches.	Thickness of Haunch, inches.	Thickness of Invert, inches.
2	3	6	6
6	7	12	7
12	13	20	13

If ground is soft or trench is unusually deep, these thicknesses must be increased according to experienced judgment.

If reinforcement is used, the thickness for conduits of ordinary sizes is usually determined by the minimum thickness of concrete which can be laid so as to properly imbed the metal. This minimum for the large diameters where steel is advisable may be taken as 6 inches.

Methods of Conduit Construction. There are four general methods of construction of concrete conduits: (1) The lower portion of the invert is laid by template and the remainder of the circle by centering. (2) The invert is formed by an inverted center, and the arch by an upright center. (3) A center the size of the entire sewer, but with a removable bottom, is placed, the sides and arch are built, and then the bottom of the center is removed, and the invert is laid. (4) The entire sewer is formed as a monolith. The size of the sewer and the character of the work influences the choice of method.

FIG. 159.—Creek Culvert at Kalamazoo, Mich. (*See p.* 504.)

If the invert is to have a brick lining or a granolithic finish, after excavating the material to the required grade and shape, profiles or templets are placed in advance of the finished concrete, and the surface is formed with the aid of a straight-edge placed longitudinally from the finished concrete to the nearest template. If the sides run up sharply, as in a small sewer, the concrete may be held in place by strips of lagging, 2-inch by 2-inch for a very small sewer, or wider for a larger size. This lagging rests at one end on the finished concrete, and at the other end on the template, and is placed as the work progresses. In horseshoe sewers the invert may be shaped with templates and straight-edge, and the side walls laid back of plank forms.

One of the simplest methods of constructing a small sewer whose invert is to be entirely of concrete, without reinforcement, is that adopted by the New York Transit Commission.* The process is described as follows:

> Previous to setting the invert form in place for constructing a length of invert, concrete was placed on the bottom of the trench in a layer thick enough to bring its top surface up to within from ½-inch to ¼-inch of flow-line grade. To insure the accuracy of this work and also to insure the accurate alignment of the form a template was suspended from the trench timbering and adjusted to line and grade. After placing the bottom layer of concrete the form (a center 12 feet in length) was accurately set in position by resting its rear end on the end of the last completed invert and supporting its forward end on a foundation accurately set in grade. The flow-line was then accurately formed by filling the space between the bottom of the form and the concrete foundation layer with a mortar of one part Portland cement to one part sand. The form was then firmly braced in position by struts nailed to the trench sheeting, and vertical planking was set up to form the outside of the spandrel. The concrete was then placed and carefully rammed against the form so as to insure a smooth surface. The invert concrete was composed of one part Portland cement, two parts sand and four parts broken stone to pass a 1-inch ring. This mixture was placed (not dropped) into position and carefully rammed. The ends of each successive section of invert were mortised to insure a firm and intimate connection with the next section, and 2 by 4-inch strips, laid longitudinally along the center of the tops of the side walls of the invert section, formed mortises for bonding the arch ring to the invert. The forms were left in place at least 24 hours to allow the concrete to set. After the invert was set and the form withdrawn a thin cement wash was brushed over its surface to smooth any slight roughness. This work gave a surface almost polished in comparison with the best brickwork.

This method of procedure affords no opportunity of troweling the surface, but in a sharply curved invert it is difficult to use a trowel. The plan

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*Engineering News, Mar. 6, 1902, p. 199.

described is not suitable for a large reinforced sewer because so much time is required to set the center and the steel that the layer of concrete in the bottom sets too hard to unite with the mortar finishing coat.

In a large conduit the smoothest and best wearing surface is obtained by laying a comparatively narrow strip of invert by means of profiles or templets and straight-edge, and troweling it. If desired, a granolithic (or mortar) finish may be given, but with thorough troweling, excellent results are secured with concrete. The arch center, which in such cases must be nearly a complete cylinder, is placed after the strip of invert concrete has set, mortar is spread on the edges of the invert strip already laid, and the circle is completed with fresh concrete. A longitudinal groove also assists in forming a tight joint.

To avoid this joint, a similar plan has been followed to that just described, except that the form, which is a complete cylinder open at the bottom, is placed, before laying any concrete, upon concrete blocks previously prepared in molds and then laid in the bottom of the trench. The lowest strip of invert is not laid until after the sides and arch are in place, the concrete for it being let down through holes left in the crown for the purpose, and troweled as thoroughly as the obstructions of the forms will permit.

It would at first appear that the sewer could more readily be made monolithic by placing a complete cylinder and pouring concrete around it for the invert arch. The objection to this, however, is the great difficulty in placing the concrete in the extreme bottom, and also the tendency of the center to "float" from the upward pressure of the concrete. This difficulty is also encountered to a less extent in the method described in the preceding paragraph.

In a sewer whose invert and arch are constructed separately, the arch centers are made and placed as for brick, except that a smoother and tighter surface is necessary, and the forms are oiled to prevent adhesion. A covering of sheet metal has often been successfully used. In order to lay the concrete of the arch sufficiently wet to obtain a smooth surface, an outside set of forms, open at the crown, is usually essential.

The laying of a large water conduit for the Jersey City Water Supply Company is illustrated in Fig. 110, page 370.

If a plaster finish is required by the specifications, the mortar may be spread upon the arch center before placing the concrete, or troweled on to the intrados after the completion of the work. In the aqueduct of the Metropolitan Water Works, Massachusetts* (see Fig. 157, p. 503), a

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*Third Annual Report, Metropolitan Water Board, 1898, p. 56.

Portland cement wash was first used on the Portland concrete arch, but it was afterwards found that thin plastering gave better results. The plastering was put on to increase the water-tightness and to make a smoother surface. As a rule, the authors do not consider it necessary or advisable to plaster the arch.

Conduit Forms. The construction of forms* so that they may be readily "struck" and removed requires considerable ingenuity and design. Invert centers for a small sewer, designed by Mr. William G. Taylor and employed in the Medford, Massachusetts, sewers, are illustrated in Fig. 160.

FIG. 160. — Center for Invert of 30-inch Sewer at Medford, Mass. (*See p.* 508.)

Conduits in Tunnel. The methods of construction, except as regards the handling of the concrete, are substantially the same in tunnel as in open-cut. It is generally necessary, however, to provide loose longitudinal lagging for the arch, and place it stick by stick as the concrete is laid. The extreme crown or key for a width, say, of 2 feet, is most easily laid

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*Various styles are referred to under "Forms" in References, Chapter XXIX.

upon cross strips or short segments in the same way that a brick arch in tunnel is keyed. The concrete for the key must be mixed fairly dry, and rammed lengthwise of the tunnel.

The tunnel section of the conduit of the Jersey City Water Supply Company is similar in inside dimensions to the open-cut section. (Fig. 158, p. 504.) It is plain concrete with no reinforcement. The thickness of the arch and sides is 8 inches and of the invert 6 inches, but points of rock are allowed to jut into this section "provided a minimum thickness of 6 inches is maintained in the arch, and of 3 inches in the sides and bottom."

TUNNELS

The general principles of design and methods of construction for large railway tunnels are similar to those for sewer and water conduits. The external strains are of course greater and must be provided for according to local conditions. In some cases water-tightness is essential; in others, which compose the large majority, the drift is through dry material, and the ballast may be laid directly upon the bottom.

Tunnel Design. The standard section of a double-track tunnel of the Pittsburgh, Carnegie & Western R. R.* has an arch 26 inches thick and side wall laid on a batter, inside, of one inch to the foot, and of such thickness as to reduce to 26 inches at the springing line.

The standard section of single arch† in the New York Subway for a tunnel 25 feet wide is 18 inches at the crown. In rock drift this thickness is carried down to the springing line, from which point the inside face is battered inward. In deep open-cut construction the arch is thickened at the haunches to about 4 feet, and the outside of the wall is waterproofed.

The East Boston Tunnel, completed in 1904, is shown in section in Fig. 161. The sketch also illustrates the general construction of steel framework and lagging which, after completion, were entirely removed. The invert between A and B was laid after the rest of the section was complete. The method of carrying on the work is described on page 511.

The approaches to the Harlem River Tunnel‡ of the New York Subway were excavated in open-cut, then roofed over, and the tube thus formed pumped out. The section of this tunnel under the river is lined with cast-iron segments.

The single-track tubes of the Pennsylvania R. R. tunnels§ under the

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**Engineering News*, May 21, 1903, p. 447.

†Contract Drawing No. C 9.

‡George S. Rice in Journal Association of Engineering Societies, Dec., 1902, p. 224.

§*Engineering News*, Oct. 8, 1903, p. 327.

channel of the Hudson River at New York City are designed with a cast iron shell made in segments bolted together and lined on the inside with concrete 2 feet thick.

Methods of Tunnel Construction. Concrete side walls and arches in tunnels constructed without the use of compressed air are laid by means of forms and centers, whose design varies with the character of the excavation

FIG. 161.—Section of East Boston Tunnel during Construction. (*See p.* 511.)

and the general arrangement of the structural machinery.* To provide clearance so that the arch center may be lowered and moved ahead, the side walls may be carried up above the springing line. For supporting the center, a temporary frame consisting of a timber resting on posts is set up close to each side wall, and the center is jacked up to line and supported by wedges. By placing the side timbers in advance, the arch may be hauled ahead on rollers by hand tackle or hoisting engine.

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*In the serial on The New York Rapid Transit Railway, *Engineering News*, Sept. 18 and Oct. 8, 1902, are excellent descriptions with sketches and illustrations of the methods of construction on one of the sections of the New York Subway and in the Harlem Tunnel. See References for further examples.

The East Boston Tunnel, shown in Fig. 161, is an interesting illustration of a tunnel entirely of concrete built with the aid of compressed air.* Two side drifts, solidly timbered, were kept from 60 to 150 feet in advance of the shield, so that the concrete side walls, which were built in these to a height of about 16 inches below the springing line of the arch, had an opportunity to set for about ten days before the shield reached them. The shield, resting on rollers, moved along on these side walls, and the main excavation was made under it. The concrete arch was built under the tail end of the shield, in lengths of 30 inches, as soon as the earth was removed. The shield was forced ahead by 16 hydraulic jacks, acting against the cast-iron cruciform push rods, 3 inches in diameter, shown in the drawing, which were placed in the concrete in 30-inch lengths, so as to form continuous rods the entire length of the tunnel. The supports for the centering consisted of steel ribs,† also shown in the figure, placed 2½ feet apart, and supporting 4-inch lagging, against which the concrete was laid. Portland cement grout, usually 1 cement to 2 fine sand, was forced in on top of the arch so as to form a film about 1½ inches thick. The invert was laid as the shield progressed. The progress of excavation and lining in May, 1901, was about 6 feet in twenty-four hours, about 60 men being then employed on each of the two shifts.

The specifications for the East Boston Tunnel‡ limited the sizes of the gravel to 2 inches, and stated that 5% only should be less than ¼ inch. The proportions required that "to each 123 pounds of dry Portland cement there shall be 2½ cubic feet of sand and 4 cubic feet of gravel, and such a proportion of water as the engineer shall from time to time determine. The sand and gravel shall not be packed more closely for the above measurements than is done by shoveling in a dry state into a measuring box." Compensation was awarded the contractor when these proportions were varied. Crushed stone screenings were largely used instead of sand.

Closing Leaks. In the East Boston Tunnel a layer of neat cement mortar was spread upon a surface of old concrete before laying a new section, but even this did not prevent slight percolation of water at these joints after the removal of the air pressure. Although the leakage through these was almost inappreciable, they gave the walls a somewhat unsightly appearance, and to stop them holes 6 inches or less in depth were drilled in the concrete, and ⅜-inch pipes inserted, through which neat cement grout was forced by a power pump. The leakage in September, 1904, in 1.4

*Howard A. Carson in Journal Association of Engineering Societies, Dec., 1902, p. 205.

†Ribs were of wood on one of the sections.

‡Construction Contract, Boston Transit Commission, Section B, East Boston Tunnel, 1900.

miles of tunnel, — over one-half mile being directly under the Harbor, — was not more than 7 to 8 gallons per minute.

SUBWAYS

Subways are technically distinguished from tunnels as constructions in open-cut instead of drift, although portions of a subway are often really of tunnel construction. The term *subway* is applied to accessible conduits for water mains, electric cables, etc., as well as to underground passages for traffic, but it will be considered here in the latter sense only.

Subway Design. To save the head-room required by a single arch, the roof of a subway may be of flat construction. The larger portion of the New York Subway is built with a framework of steel I-beams. The bents are spaced about 5 feet apart and the roof is formed by arches of concrete* sprung between the lower flanges of the cross girders. The concrete also covers the steel so that it is completely imbedded. The walls are also of concrete about 15 inches thick, forming arches between and imbedding the posts. A portion of the subway is of reinforced concrete construction with 3-inch angles, and 1⅛-inch and 1¼-inch rods imbedded, as illustrated in Fig. 162, which is arranged† from the original drawings.

The question of waterproofing is an extremely important one in subway construction, especially where the grade is below the level of ground water. Methods adopted in the New York Subway are described more fully on page 423.

During the course of construction in New York it was decided to widen one of the portions already complete. The contractors moved the concrete side walls and roof, 275 feet long, bodily, without injury.‡

ARCH BRIDGES

Concrete arch bridges, even from an architectural standpoint, may compare favorably with cut stone masonry, and the cost is usually much lower. In comparison with steel, concrete is more durable and presents a finer appearance. Referring to the comparative cost, Mr. Joseph R. Worcester§ says:

> The speaker's experience is, that, in the case of a highway bridge of moderate span, a comparison between a steel-concrete bridge and one of steel with buckle-plate floor filled on top will show that the steel-concrete

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*Concrete has superseded brick for such arches.

†With permission of Mr. William Barclay Parsons, Chief Engineer.

‡See descriptions and illustrations in *Engineering News*, June 11, 1903, p. 515.

§Transactions American Society of Civil Engineers, Vol. XLVI, p. 101.

bridge is considerably cheaper; and that where good foundations are easily obtained, if the total cost of bridge and abutments is included in the comparison, there is a fair chance for steel-concrete construction to compete with all-steel.

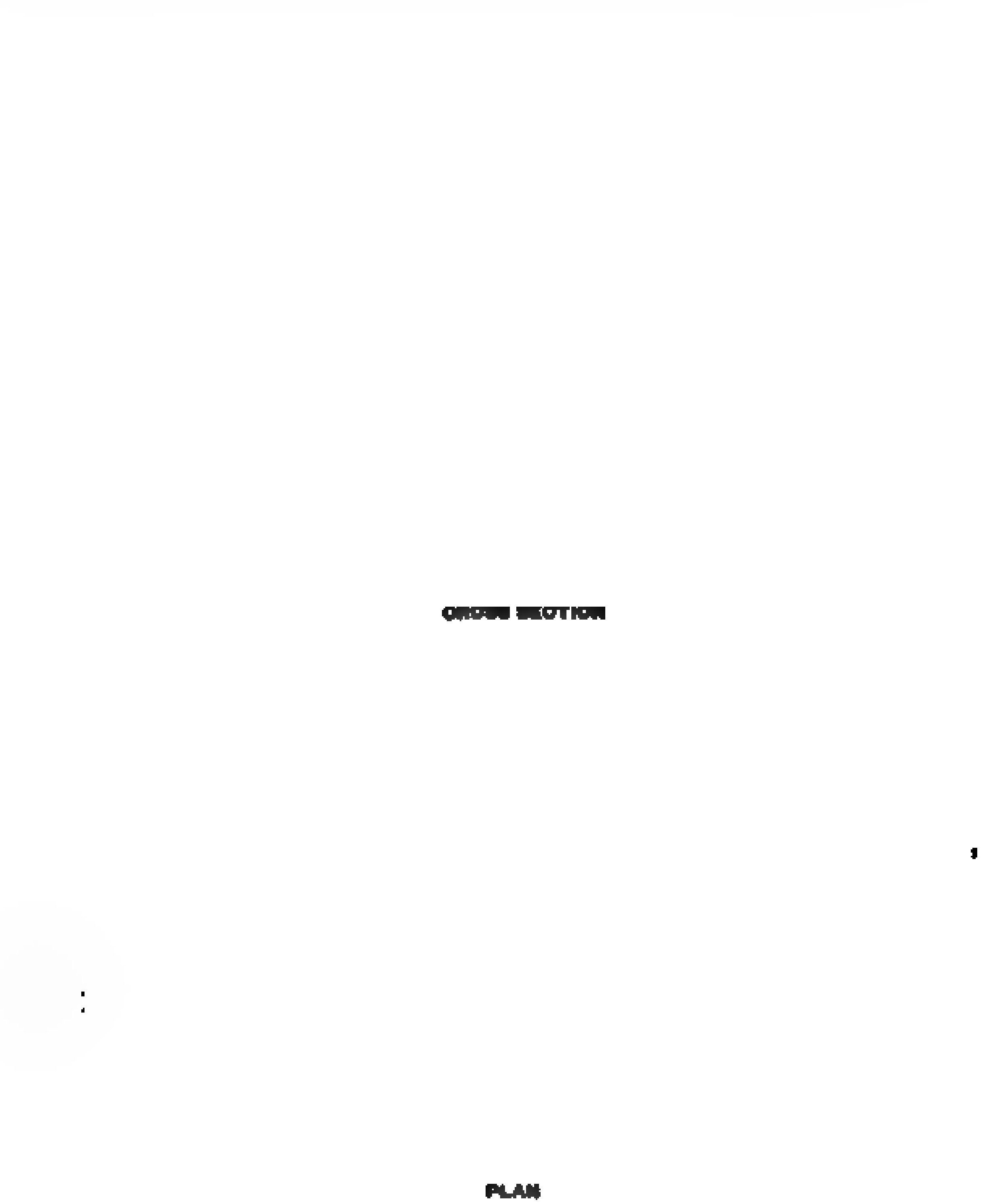

Fig. 162. — Typical Section of Reinforced Concrete Construction in New York Subway. (*See p.* 512.)

The use of concrete for bridge work developed earlier in Europe than in the United States.* Especially noticeable in the different designs which

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*In References, Chapter XXIX, will be found a list of differing types which will serve as illustrations of European and American design.

have been constructed is the great variation in the thickness of the arch and in the quantity of steel employed. This is due not only to differences in strains, but also to the different factors of safety required and to the limited knowledge of many designers as to the actual strength of concrete in structures of this class.

The adaptability of concrete to bridge work is illustrated in the notable examples of concrete arches built within recent years.

The Big Muddy River Bridge* of the Illinois Central Railway has three spans, each a concrete arch with no reinforcement, 140 feet between abutments. It is designed as a masonry arch, consisting of separate blocks or voussoirs, and is built in the same fashion, blocks being molded at different positions in the ring and allowed to set before laying the intermediate ones.

The Zanesville, Ohio, bridge†, designed by Mr. Edwin Thacher, is composed of eight reinforced concrete arches, the largest being 122 feet span and 11½ feet rise, with a thickness at crown of 30 inches. The arches are each reinforced with fifteen pairs of steel bars, which are ¾-inch thick by 5 inches wide for the 122 feet span, and narrower for the shorter spans. These bars are placed in two layers, one 2 inches from the intrados, and the other 2 inches from the extrados.

The Châtellerault bridge, France,‡ has a central arch of 164 feet span and 15.75 feet rise, and two side arches of 131 feet span. The thickness at the crown is about 21 inches. Each arch is formed by four separate arched beams about 20 inches wide, heavily reinforced by the Hennebique System, with round rods. The floor is also of reinforced concrete, and is carried by iron posts which rest upon the steel-concrete arch.

At Laibach, Austria,§ is a three-hinged arch bridge, designed by Prof. J. Melan, 108 feet span and 14.6 feet rise, having a thickness at crown of 20 inches, and reinforced with steel lattice girders. Relieving arches of concrete support the floor.

Examples of other bridges are given in References, Chapter XXIX.

Design of Arch Bridges. Flat bridges consisting of straight reinforced girders, connected by reinforced floor slabs, are designed according to the principles of girder and floor design, given in Chapter XIV. The distribution of the loading is of course the same that would be chosen for any other type of bridge in the same location.

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*H. W. Parkhurst, *Engineering News*, Nov. 12, 1903, p. 423.

†For full description, see Concrete-Steel Arch Y-Bridge at Zanesville, Ohio, *Engineering News*, March 27, 1902, p. 261.

‡*Revue Generale des Chemins de Fer*, Dec., 1901.

§A Concrete-Steel Three-Hinged Arch Bridge, *Engineering News*, July 16, 1903, p. 61.

In Europe the three-hinged type of design has been quite largely employed, but in the United States monolithic construction is the usual style.

Most concrete arches are reinforced with steel, although some engineers claim that in many cases there is no advantage in reinforcement if the intrados curve is properly chosen. However, while an arch of plain concrete can be safely built, the steel reinforcement adds to the safety, and permits the use of less material by resisting the bending moments which tend to produce tension under eccentric loading.

An arch of plain concrete is usually designed by the same methods employed for a stone masonry arch, the line of resistance being found graphically. The elevation of the arch is arbitrarily divided by radial joints into blocks of such length that the line of resistance does not depart very far from a curve. For an arch of 60 feet span a convenient length of block is 5 feet. The method of drawing the line of resistance is given in various treatises on Mechanics.* The theory of the elastic arch, which presents a more scientific treatment for a monolithic arch, is thoroughly treated in Greene's "Mechanics of Engineering," also in Howe's "Treatise on Arches."

A reinforced arch also is most commonly divided by arbitrary joints into separate blocks, and the line of resistance graphically located. The unit pressure and the bending moment at each assumed joint is then calculated. Except in three-hinged arches, the effect of temperature, which may produce nearly as high a stress as the bending moment, should not be neglected. Having drawn the line of resistance in the customary manner, the bending moment at the different sections is calculated from the assumed positions of the loads upon the arch, and by combining these forces the stresses are calculated by the usual principles of reinforced concrete design.

The maximum stress, then, at any plane, is due to the thrust of the arch acting along the line of resistance; the bending moment caused by the loads arranged in such a manner as to produce the greatest possible stress upon the plane; and the bending moment due to temperature changes. The direct thrust is borne partly by the concrete and partly by the steel.

As the tensile stress in the concrete is comparatively low, some engineers assume that the concrete bears a small portion of the tension. In the light of recent experiments, there appears to be no good reason for this unless the total pull is so small that the stretch allowable in plain concrete is not ex-

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*Mr. H. W. Parkhurst presents an excellent stress diagram of a plain concrete arch in *Engineering News*, Nov. 12, 1903, p. 425.

ceeded. (See p. 287.) Formulas in which the tension in the concrete is taken into account are presented in Appendix II.

Reinforcement in the top of the arch resists negative binding moments which under certain loading may produce tension there, and under normal loading also aids the concrete in bearing compression.

Methods of Arch Construction. There are two general methods of laying the concrete in an arch, each of which have strong advocates. By the first, the arch is laid in separate blocks across the bridge, and by the second, in narrow ribs from abutment to abutment. If the block method is followed, the lowest stones at the springing line are laid first, then stones intermediate between the spring and the key, next the two stones each side of the key, and finally, after filling in the intermediate blocks, the key is placed. This distributes the weight of the concrete uniformly over the arch center, and prevents unequal settlement, which tends to crack the arch near the springing lines. On the other hand, the entire weight falls upon the center, and the latter must be very strongly built. The arch thrust acts at right angles to the joints, and as the blocks extend clear across the bridge, there is no danger of longitudinal splitting, but the radial joints offer planes of weakness in bending.

By the other method the work can be readily arranged so that a day's labor consists of the laying of a single rib, thus forming a complete arch of itself, which as soon as it sets bears its own weight. This arch section has no joints, so that when subsequently loaded the bending moment is best resisted.

A small arch, where the center can be solidly built, may be laid at one operation, commencing at both abutments and working toward the key so that it is in fact a monolith.

The spandrel or face walls may be carried up at the same time the arch ring is laid, or may be connected with it later by leaving short lengths of steel projecting radially from the concrete of the arch.

If steel is introduced, the consistency of the concrete must be wet enough to thoroughly coat it. This may be accomplished by a quaking or jelly-like mixture, which requires but slight ramming.

From an architectural point of view, the treatment of the face is of much importance. For a discussion of the different methods reference should be made to page 380.

Railings and ornamental work may be cast in molds (see p. 470) and put in place after hardening.

Arch Centers. In designing and building the center for a stone arch, it is especially important to provide sufficient stiffness to prevent its distortion

before the keying of the arch. This tendency may be partially overcome by following the methods described in the preceding paragraphs.

The design of the center is similar to that for stone masonry except that the lagging must be carefully dressed material placed close or covered with sheet metal. In connection with the description of arch centers which he has built, Mr. James W. Rollins, Jr., gives the following notes:*

> For small arches the simplest center is a circular rib made of three pieces of 2-inch plank, laid with broken joints, all being spiked solidly together, with a tie of plank at the springing. On this, 1-inch lagging is laid close. For a larger arch, the circular rib, as above described, with generally three braces, one at center and one on the quarter at each side, is used, the center of the whole rib having a post under it. We have used such a center up to 30-foot span for both brick and granite arches, carrying a 30-inch arch sheeting.
>
> The design of a center for larger arches depends upon local conditions, also upon the relation of rise to span. In flat arches, with low side walls, it is well to use posts with intermediate bracing, on numerous supports. In a high arch we may use long braces extending directly from a center support to the rib, at intervals of 6 feet to 8 feet.

Mr. Rollins advocated for wedges, seasoned oak, 8 inches wide, 4 inches thick at the thick end, 2 inches at the thin end, and 18 inches long, planed on sliding faces, and thoroughly greased. When setting the center, these wedges, placed between the caps on the bents and the corbels under the lower chord of rib, are tacked together to prevent slipping.

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*Journal Association of Engineering Societies, July, 1901, p. 10. For examples of centers built in various places, see References, Chapter XXIX.

CHAPTER XXVII

RESERVOIRS AND TANKS

A new field is being developed for concrete design in the building of covered reservoirs and filtration plants for water purification works. Plain or reinforced concrete is now commonly employed for the floors, columns, vaulted roofs, tanks, and filter basins. The Filtration Works at Little Falls, N. J.,* furnish a modern example of such construction. For open reservoirs, concrete is frequently substituted for stone masonry both in the retaining walls and core walls, and also is used for lining the bottom.

Concrete tanks are used not only for water but for chemicals.

OPEN RESERVOIRS

The principles of design and construction of retaining walls have already been discussed in Chapter XXV. The contraction cracks, which are almost certain to occur in long walls of any class of masonry, may be provided for by some form of expansion joint. Cut-off walls of clay† may be placed to prevent the passage of water through these vertical joints, or open wells‡ may be left at intervals in the walls, and after setting for a month or more filled with concrete. This concrete filling is placed preferably upon a cold day, when the contraction in the wall is greatest.

The lining for the bottom depends upon the character of the underlying soil or rock. Usually a layer of 1 : 2½ : 5 concrete 4 to 8 inches thick, if properly laid and troweled, will provide a lining sufficiently impervious for practical purposes.§

In small reservoirs, where earth and rock meet so as to present danger of unequal settlement and consequent serious leakage, a strip of reinforcing metal may be placed over the line of division.

COVERED RESERVOIRS

A common type of design for covered reservoirs consists of a concrete bottom, underlaid, where necessary, with 12 to 16 inches of clay puddle

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*Transactions American Society of Civil Engineers, Vol. L, p. 394.

†See paper by Chas. W. Paine in Journal Association of Engineering Societies, October, 1902, p. 151.

‡Transactions American Society of Civil Engineers, Vol. L, p. 406.

§For other methods of lining see Chapter XX on water-tightness.

and laid in the form of inverted groined arches. Piers of concrete or brick rest upon the thick haunches of the arches, and the roof is formed of groined arches supported by the piers and covered with a layer of earth. For the prevention of leakage, the principles already discussed in Chapter XX, on Water-tightness, are applicable. The contraction of the concrete is a common source of cracks, but when comparing concrete with other kinds of masonry, it must be noted that concrete is no more liable to temperature contraction than brick and stone, the brick division walls, for instance, of the Albany Filtration Plant,* showing cracks similar in number and appearance to the cracks in the outside concrete walls.

Reservoir Walls.† Since the walls are supported at the top by a roof, there is less danger of overturning, and thinner sections may be used than for open reservoirs. This class of structure also presents opportunity for thin walls reinforced with steel.

Walls of plain concrete for shallow reservoirs or filter beds are frequently 2 feet to 2 feet 6 inches at the top, with a batter on the outside of 1 in 10.

The wall of a circular reservoir supporting a dome-shaped roof should be reinforced at the top with one or more rings of steel to resist the thrust.

Methods of forming expansion joints for open reservoir walls, described on page 518, are also applicable to covered reservoirs.

Reservoir Piers. The dimensions of the piers are readily calculated after designing the roof and determining its weight, and the weight of the earth covering. In concrete piers of dimensions suitable for reservoirs, a working pressure of 350 pounds per square inch may be safely allowed when the proportions of the concrete are 1: 2½: 5.

A floor of inverted groined arches will distribute the pressure of the piers if the soil is unstable. In some cases it may be necessary to place reinforcing steel in the footing (see design of column footings on page 478) to prevent unequal settlement.

In ordinary cases no reinforcing steel is needed in the piers. However, if the load upon them is extra heavy and the reduction of their dimensions is of importance, steel may be introduced to assist in carrying the compression. (See p. 328.) Also, if the columns are of considerable height, say, over 20 feet, a small rod near each corner, with occasional horizontal hoops, may be placed as described on page 466.

Reservoir Floors. The floor should be smooth, fairly impervious, and

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*Transactions American Society of Civil Engineers, Vol. XLIII, p. 282.

†Methods of calculating the wall pressure, the amount of reinforcement required, as well as other tables and data relating to covered reservoir construction, are given in a paper on Covered Reservoirs and Their Design, by Freeman C. Coffin in Journal Association Engineering Societies, July, 1899, p. 1.

strong enough to resist the upward water pressure from the underlying soil when the reservoir is emptied. Mr. Coffin* considers a thickness of 3 or 4 inches sufficient when the soil is so compact that there is no danger, when empty, of pressure from without. In pervious earth he suggests 6 inches of concrete for heads as great as 20 feet.

Inverted groined arches for the floor not only distribute the pressure of the piers, but also present increased thickness of concrete around the piers where there is most danger of unequal settlement, give a minimum volume of concrete, and afford channels for the passage of the water when the reservoir is emptied.

The groined arches are laid in alternate diamonds before the piers are built, so that each pier will rest upon the corners of four diamonds. The method of laying the floor arches at the Albany Filtration Works† is illustrated in Fig. 163.

FIG. 163. — Reservoir Floor. (*See p.* 520.)

Before the concrete has set, the surface may be covered with a granolithic or mortar finish, as in sidewalk construction (see p. 442), or it may be simply troweled. Methods of treating joints between blocks and other means of waterproofing are discussed on page 424.

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*See second footnote on p. 519.

†Allen Hazen in Transactions American Society of Civil Engineers, Vol. XLIII, p. 262.

Reservoir Roofs. Groined elliptic arches* are especially suited to reservoir roofs because requiring the minimum volume of concrete to support their own weight and the weight of the earth above them.

Mr. Coffin† says that the cost per cubic yard of groined arches of concrete is about one-half that of brick masonry. Although the centering costs more than brick because a tight surface is necessary, the brickwork is more expensive on account of the great amount of cutting required. He further states that "the cost of the centering, their supports, placing and removing them, is from 15 to 20 cents per square foot for the interior surface of the reservoir if it is all centered at once."‡

Mr. Leonard Metcalf has compiled a table§ of data relating to reservoirs in the United States covered with groined arches, which shows a range in span of arch from 10 feet 6 inches to 16 feet, a rise varying from one foot 6 inches to 4 feet, and a thickness at crown, in all cases but one, of 6 inches. The proportions of the concrete range from 1 : 2½ : 4 to 1 : 3 : 5.

TANKS

Reinforced concrete is cheaper for tanks than sheet steel, and more durable than wood. It is especially adapted for tanks used in paper and pulp mills to hold chemicals. When made of wood or other material which is affected by acid and bleach liquor, such tanks require constant repairs. Concrete not only furnishes a durable material, but one into which outlet castings may be readily built, and to which, if properly flanged so that the concrete cannot shrink away from the metal, the cement will adhere and form a tight joint. The gates and other connections, which are usually of brass or bronze, must be so heavy that the corrosion and wear upon them will not necessitate removal and therefore repairs to the concrete, since it is impossible to form a satisfactory joint between old and new concrete in a thin wall.

There are two distinct methods of concrete and mortar tank construction. In one, forms are built and the concrete is laid with metal reinforcement in the usual manner, and in the other, a framework of metal lathing, the shape of the tank, is constructed, and Portland cement mortar plastered upon it, as described on page 469.

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*Methods of centering and placing the concrete of the vaulting are described in detail and illustrated in Mr. Hazen's paper in Transactions.

†See second footnote on p. 519.

‡Mr. Coffin also gives interesting diagrams showing quantities and costs of materials and labor for covered reservoirs.

§See Report of Annual Convention of the New England Water Works Association, 1903, *Engineering News*, September, 1903, p. 238.

Methods of Construction. The materials for the concrete must be very carefully proportioned so as to give a water-tight wall (see p. 417), and the stone should be of such size that a good surface can be readily obtained. The concrete should be mixed so wet that it will completely cover the metal reinforcement and flow against the form, and it is absolutely essential that the entire tank be built in one operation.

Mr. William B. Fuller's methods of constructing a thin wall require that the concrete be mixed very wet, so that after wheeling 25 feet it will settle down to a level in a wheelbarrow. The laborer shovels it from the barrow, throwing one shovelful in a place, and goes the entire length of the section or around the circumference, thus forming a very thin layer and preventing the separation of the ingredients.

The forms for the Little Falls tank described and illustrated on page 523 consisted of 2½ by ⅞-inch tongued and grooved boards, planed one side and placed vertically. Around the outside of the top of this cylinder of boards was placed a horizontal rib consisting of two sets of boards, 8 in each set, cut to a circle and laid in two thicknesses so as to break joints. Below this rib, a wire rope was wrapped around the forms spirally, so that the separate spirals were about one foot apart. The lower ends of the staves were held by the bottom portion already built, otherwise another rib would have been required at the bottom. The inside form consisted of three cylindrical centers built like ordinary sewer centers and placed upright one above the other, each about one foot 3 inches high. These were suspended so that the bottom of the lowest allowed for the 3-inch thickness of the concrete bottom. They were held temporarily in place sideways by pieces of board 3 inches long placed between them and the outside forms. As soon as the centers were fixed in position the concrete for the bottom was poured down through the middle of them and immediately afterward the walls were poured. This concrete flowed out slightly under the bottom center, but was easily removed after setting. There were no reinforcing angles between the bottom and the sides. The rods of the bottom extended very nearly to the outside lagging, and the side rods extended down almost to the lower surface of the concrete bottom. Two tanks were built at once, and the contract price of each was $100.

Examples of Tanks. The Filtration Plant at Little Falls, N. J., whose structural features were designed by Mr. Fuller, has a tank or well 41 feet high and 10 feet in diameter, which sustains the pressure of water either from within or from without. The walls are 15 inches thick at the bottom and 10 inches thick at the top. Rings of ½-inch Ransome twisted steel rods were placed about every 2 feet in the center of the wall, and vertical

rods ¾ inch in diameter and about 5 feet apart were also set in the center of the wall, thus forming a series of hoops and posts.

On a platform in the same building is a tank 4 feet high and 4 feet in diameter. The walls are 3 inches thick, and contain rings of ¼-inch twisted rods placed about 6 inches apart, and ½-inch vertical rods about 2 feet apart. The floor of the tank is also 3 inches thick, with ⅛-inch rods spaced so as to make a 6-inch square mesh. This tank is shown in section in Fig. 164.

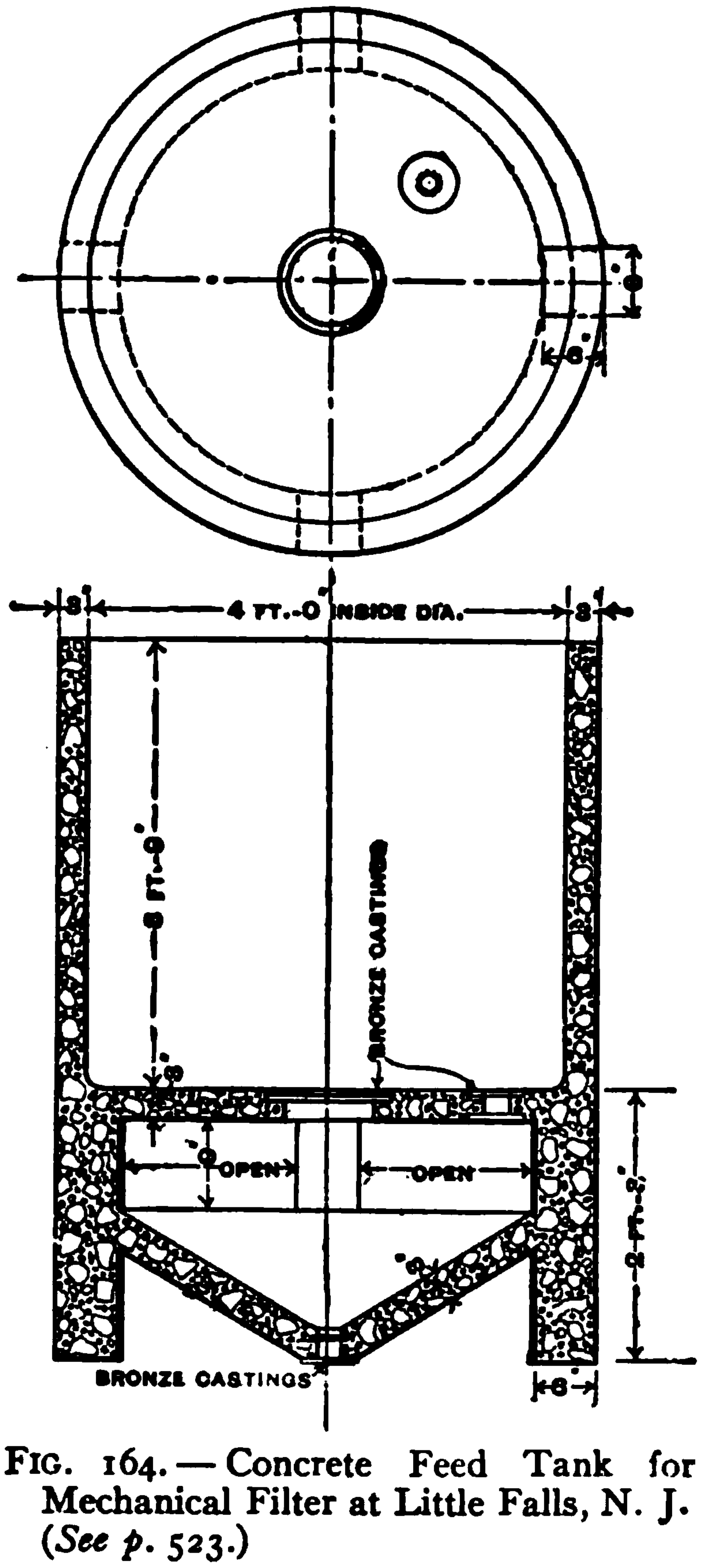

Fig. 164. — Concrete Feed Tank for Mechanical Filter at Little Falls, N. J. (*See p.* 523.)

The Illinois Steel Company, South Chicago, employ circular concrete tanks* for storing cement. These are 25 feet in diameter and 50 feet high, with walls 7 inches thick at the bottom and 5 inches thick at the top. The concrete is reinforced by rings spaced 4 inches apart and varying in diameter from one inch at the bottom to ⅜ inch at the top.

At Milford, Ohio, is a stand-pipe† of reinforced mortar 80 feet high and 15½ feet outside diameter. The thickness of the shell for the lower 30 feet is 9 inches, for the next 25 feet, 7 inches, and for the remaining 25 feet, 5 inches. The outside face is vertical. The concrete foundation is 20 feet in diameter and 6 feet thick. On top of this, T-bars, 1 by 1 by ⅛ inch, were placed radially from the center to within 6 inches of the outer edge, and the shell was started directly from these. The horizontal base around and within the shell was then strengthened by a layer of 1:3 mortar 6 inches thick in the interior of the tank and 16 inches thick around the outside of it. The shell is of 1:3 mortar reinforced with T-bars 1 by 1 by ⅛ inch, spaced 18 inches apart vertically and in horizontal rings varying from 2 inches

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**Engineering News*, August, 1902, p. 148.

†See *Engineering News*, Feb. 1904, p. 184.

apart at the base to 3 inches apart at the top. It is evident from the discussion on page 173 that a substitution of stone for a part of the sand in the mortar for this tank would have produced a denser mixture, and at the same time would have permitted the use of leaner proportions with a consequent reduction in cost.

CHAPTER XXVIII

CEMENT MANUFACTURE

This chapter contains a short historical sketch followed by a brief outline of the processes of modern cement manufacture, illustrated with views of typical machinery.

HISTORICAL

Lime must have been used by the Egyptians thousands of years before Christ, as the stones in the pyramids apparently were laid in mortar of common lime and sand. It is even thought by some that these ancients understood the principle of mixing lime and clay together to make a real cement.

Concrete was made by the Romans as early as several centuries before Christ. For most of their work, they used lime mixed with sand and stone, but understanding the value of puzzolana or volcanic ashes to render lime hydraulic, they employed these two materials in combination with the sand and stone for marine construction. For less important work, they often mixed lime and coarsely powdered brick with the aggregate. Vitruvius, writing in the first century, describes methods of making concrete with lime alone, and also gives as the formula for making it of slaked lime and Italian puzzolana:

12 parts of puzzolana, well pulverized.
6 parts of quartz sand, well washed.
9 parts of rich lime, recently slaked; to which is added
6 parts or fragments of broken stone, porous and angular, when intended for a "pise" or a filling in.

In the Middle Ages concrete was employed, after the Roman fashion, for both walls and foundations. In the former it was generally laid as a core faced with stone masonry. Large stones were often imbedded in the mass.

The fact that clay contained in certain limes rendered them hydraulic was discovered by John Smeaton, when studying the designs for the third Eddystone Lighthouse, about 1750. Early in the following century, Vicat, by his extended scientific researches in France, earned for himself the name of the founder of hydraulic chemistry.

In England, in 1796, James Parker made from nodules of argillaceous limestone, calcined and ground, what he called Roman cement. This process he patented, and from it the Natural cement industry was developed. It was Joseph Aspdin, of Leeds, England, who really invented Portland cement by discovering in 1824 that an artificial mixture of slaked lime and clay, highly calcined, formed a hydraulic product. On account of its resemblance in color and hardness to the Portland stone which was much used in England at that time, he called his invention Portland cement. Two patents had been granted in England a few years before his time, but as in these the materials were not heated to vitrification, hydraulic lime instead of cement was produced.

The Portland cement industry was not developed to any great extent until about twenty years after Aspdin's discovery, when J. B. White & Sons in Kent, England, commenced its manufacture. Later, Mr. John Grant gave a great impetus to Portland cement manufacture by experimental studies upon the practical action of cements, mortars and concretes under varied conditions. The results of his tests he presented to the Institution of Civil Engineers in 1866, 1871, and 1880.

The first manufactory for producing Portland cement in France was established toward the middle of the last century at Boulogne-sur-Mer. In Germany the first factory was erected soon after this, for the production of the Stettin Portland cement, and with such successful results that in 1900 Germany produced more Portland cement than any other country.

The discovery in the United States of a rock suitable for Natural cement was made in 1818 by Canvass White, an engineer connected with the construction of the Erie Canal, and Natural cement was made in Madison and Onondaga Co., N. Y., in that year. The first Natural cement in the Rosendale district was made at Rosendale, Ulster Co., N. Y., about 1823. Mr. D. O. Saylor was the founder of the Portland cement industry in the United States. His discoveries were made in the Lehigh Valley in 1875 and his first factory was built about 1878.

PRODUCTION OF CEMENT

The total production* of hydraulic cement in the United States for 1903 was 29 899 140 barrels, of which 22 342 973 barrels were Portland cement, 7 030 271 barrels were Natural cement, and 525 896 barrels were Puzzolan or Slag cement. The average values per barrel were, for Portland cement $1.24, for Natural, $0.52, and for Puzzolan, $1.03.

The superior quality of Portland over Natural cement and the increasing

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*L. L. Kimball in Mineral Resources of the United States, 1903.

economy of its manufacture is evinced by a comparison of these figures with those of 1890, when only 335 500 barrels of Portland cement were produced against 7 082 204 barrels of Natural cement. The imports of cement in 1890 were 1 940 186 barrels, and in 1903, 2 317 950 barrels.

The production of Portland cement in the United States by individual States is represented in the following table:

Production of Portland cement in the United States in 1900 *and* 1903*

State.	1900.			1903.		
	Number of works.	Quantity.	Value, not including packages.	Number of works.	Quantity.	Value, not including packages.
		Barrels.			Barrels.	
Alabama[1]				1		
Arkansas[2]	1	40 000	$70 000	1		
California	1	44 565	89 130	3	631 151	$1 019 352
Colorado	1	35 708	71 416	1	258 773	436 535
Georgia[1]				2		
Illinois	3	240 442	300 552	5	1 257 500	1 914 500
Indiana	1	30 000	37 500	3	1 077 137	1 347 797
Kansas	1	80 000	100 000	1	1 019 682	1 285 310
Michigan	6	664 750	830 940	13	1 955 183	2 674 780
Missouri				2	825 257	1 164 834
New Jersey	2	1 169 212	1 169 212	3	2 693 381	2 944 604
New York	8	465 832	582 290	12	1 602 946	2 031 310
North Dakota	1	400	1 200			
Ohio	36	534 215	667 769	8	729 519	998 300
Pennsylvania	14	4 984 417	4 984 417	17	9 754 313	11 205 892
South Dakota[3]	1	38 000	76 000	1		
Texas[4]	2	26 000	52 000	2		
Utah[3]	1	70 000	175 000	1		
Virginia	1	58 479	73 099	1	538 131	690 105
West Virginia[1]				1		
Total	50	8 482 020	9 280 525	78	22 342 973	27 713 319

[1] Product combined with that of Virginia.
[2] Combined with Missouri.
[3] Combined with Colorado.
[4] Combined with Kansas.

About 55% of the total production in 1903 was in the Lehigh Valley of Pennsylvania and New Jersey. In 1902 63% came from that district.

PORTLAND CEMENT MANUFACTURE

Portland cement is made from a mixture of calcium carbonate and silicate of alumina.

The processes of manufacture differ with the natural state in which

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*Data from Mineral Resources of the United States, 1902 and 1903.

these materials are found, but the operation consists essentially of (1) pulverizing and mixing the two ingredients, (2) heating to a temperature which is near the melting point, *i. e.*, calcining, (3) grinding to a fine powder.

If either of the raw materials occurs in a moist state it is generally customary to mix them wet, and after a preliminary grinding introduce them into the kilns. Dry raw materials for calcining or burning in the old style stationary kilns must be formed into plastic bricks with the aid of water, but the rotary kiln, invented in 1885 by Mr. Frederick Ransome, has revolutionized the manufacture of Portland cement by making it possible to introduce the mixed substances into the furnace, in either a dry or wet state, without hand labor.

After calcination, the methods of grinding the clinker are independent of the character of the raw materials or the type of kiln.

The Association of German Cement Manufacturers, to protect the good name of German Portland cement, requires that its members shall sign the following "Declaration"*:

"(*a*) The undersigned members of the Association of German Cement Manufacturers bind themselves to produce under the name of Portland cement only such an article as is made by calcining a thorough mixture, consisting essentially of calcareous and clayey substances, and then grinding the same to the fineness of flour.

"Any article made in a manner differing from the above method, or to which during or after burning any foreign substances have been added, is not recognized by them as Portland cement, and the sale of such products under the designation 'Portland cement' is regarded by them as defrauding the purchaser. This declaration does not apply to such minor additions as are made to regulate the setting time of Portland cement, and which are permitted to an extent of 2 per cent.

"(*b*) A member acting contrary to the obligations assumed under (*a*) shall be disqualified from membership in the Association, and his disqualification shall be made publicly known.

"(*c*) In making this declaration the undersigned members recognize that the officials of the Association are in duty bound to see that the assumed obligations are adhered to."

Raw Materials for Portland Cement Manufacture. The raw materials, as stated above, consist essentially of calcium carbonate and silicate of alumina. Their exact proportions are determined by their chemical composition. A usual ratio is about 75% carbonate to 25% silicate. The two substances occur in nature in so many forms that we have a

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*Max Gary in Transactions American Society of Civil Engineers, Vol. XXX, p. 8.

large range of choice in raw materials. The following combinations are actually used in different cement manufacturing plants in the United States:

Cement rock and limestone
Limestone and clay.
Limestone and shale.
Marl and clay.
Chalk and clay.
Limestone and slag.
Alkali waste and clay.

Cement rock is an argillaceous limestone, rather soft in texture, which in the Lehigh Valley usually requires from 10% to 20% of limestone to give it the correct Portland cement composition. Occasional deposits are found which are suitable to use with no admixtures, or from which the desired proportions may be obtained by mixing two different strata in the same quarry. Several other States, among them the Virginias, Alabama, Colorado, and Utah, have a geological formation similar to that in the Lehigh Valley from which Portland cement is made.

In the Hudson River Valley, near Catskill, New York, are situated large manufactories employing a hard limestone which is nearly pure carbonate of lime, requiring 20% to 25% clay or shale and producing a fine quality of cement. A somewhat similar mixture is used in California and in scattered localities in the Central States.

The marl used for cement is a wet, calcareous earth, in some localities of organic origin from shell deposits, and in other places of chemical formation. There are large cement plants using marl and clay in western New York, Ohio, Indiana, and Michigan.

Chalk and clay deposits resembling those in England are worked in South Dakota, Texas, and Arkansas.

Certain blast furnace slags similar to those used in the manufacture of Puzzolan cement, when combined with a suitable admixture of limestone, produce, after calcination, a true Portland cement.

The waste from the manufacture of soda, when employing the ammonia soda process with suitable raw materials, is substantially a precipitated chalk, and is burned with clay to produce Portland cement.*

In Germany the Alsen and Stettin brands are made from chalk and clay, the Dyckerhoff and Mannheimer brands from limestone and clay, while the Germania and Hanover works use marl and clay. In England

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*B. B. Lathbury, *Engineering News*, June 7, 1900, p. 372.

raw materials consist principally of chalk and clay. Belgium manufacturers use chalk and clay, and a Portland cement from natural rock is also manufactured in that country. In France, marl and clay, and chalk and clay, are the chief raw materials for true Portland cements.

The character and proportioning of the raw materials and the processes of chemical combination are discussed by Mr. Spencer B. Newberry in Chapter VI.

The following table illustrates the composition of various classes of materials which are used for Portland cement, and also the resulting analysis of the cement in each case:

Comparative Analyses of Raw Materials and Portland Cements.

		Cement Rock and Limestone.			Limestone and Clay.[4]			Marl and Clay.			Chalk and Clay.*		
		Cement Rock.[1]	Limestone.[2]	Cement.[3]	Limestone.	Clay.	Cement.	Marl.[5]	Clay.[6]	Cement.[7]	Chalk.[8]	Clay.[9]	Cement.[10]
Silica	SiO_2	19.06	1.98	19.92	3.30	55.27	21.50	1.75	62.10	22.52	0.35	60.30	22.10
Alumina	Al_2O_3	4.44	0.70	9.83	1.30	28.15	10.50	1.57	20.09	6.69	0.75	11.07	11.32
Iron Oxide	Fe_2O_3	1.14		2.63					7.81	3.54		8.13	
Calcium Oxide	CaO	38.78	53.31	60.32	52.15	5.84	63.50	49.24	0.65	63.82	54.95	4.40	60.76
Magnesian Oxide	MgO	2.01	0.97	3.12	1.58	22.5	1.80	0.44	0.96	0.69		1.27	1.10
Sulphuric Acid	SO_3			1.13	0.30	0.12	1.50	0.15	0.49	0.98		2.50	1.40
Carbonic Oxide	CO_2	32.66	42.94		40.98			39.16	8.00		43.17	7.47	1.94
Water	H_2O				8.37								
Organic Matter								7.50				4.06	
Other Constituents							0.40			1.08	0.85	0.45	1.38

NOTE.—Carbonates in raw materials, given in some of the analyses, have been transformed into oxide.

[1] Cement Rock. Lehigh Valley District, Penn. 21st Annual Report, U. S. Geological Survey. Pt. 6, p. 404.

[2] Pure Limestone, Lehigh Valley District. W. E. Snyder, Analyst.

[3] Lehigh Valley Cement. Booth, Garrett & Blair, Analysts.

[4] Hudson River Valley. Mineral Industry, Vol. 6, p. 97.

[5] W. H. Simmons, Analyst, 22d Annual Report, U. S. Geological Survey, Pt. 3, p. 650.

[6] Shale. Mineral Industry, Vol. 6, p. 99.

[7] Michigan. W. H. Simmons, Analyst, 22d Annual Report, U. S. Geological Survey, Pt. 3, p. 680.

[8] Water, 23%. Analysis from David B. Butler, England.

[9] Estuary Mud. Roughly dried, lost 33⅓%. Analysis from David B. Butler, England.

[10] English Portland Cement. Analysis from David B. Butler, England.

Processes in Portland Cement Manufacture. The method of mixing the materials in preparation for their introduction into the kilns has led to

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*The authors are indebted for these analyses of chalk and clay to David B. Butler, of England, who prepared them for this Treatise.

a classification of processes into (1) wet process, and (2) dry process. The former is often subdivided into wet and semi-wet, depending upon the quantity of water added at the time of the mixing.

The *wet process* is employed with soft or wet materials, such as chalk and clay, or marl and clay. The carbonate of lime and the clay are mixed in a vat or wash-mill with a large excess of water. Agitators break up the lumps and so finely reduce the particles that they are held in suspension in the water and flow off over the top of the vat. In another basin the stuff is allowed to settle, the water is drawn off, and the "slurry" becomes hard enough to handle in barrows and then form into bricks to be dried, and finally calcined in stationary kilns.

By using a smaller quantity of water, say 40 or 45%, the settling process and consequent hand-labor is avoided, and the material is made only fluid enough to handle in pumps. After grinding, it may be pumped directly into the rotaries, or, if stationary kilns are used, the pumps throw it to the drying room to be made into bricks. This process is called in England the semi-wet process, but as it is practically the only wet process used in the United States, it is here simply termed the wet process.

The *dry process* was first used in Germany as a result of the substitution of limestone for the chalk of England. The two ingredients are ground and mixed in a dry state. If the kilns are stationary, the mixed material must be moistened with sufficient water to form plastic bricks, which are then dried, but for rotary kilns no water is added, the mixture of dry materials passing, after being ground, directly into the kiln.

Dry Process with Rotary Kilns. The introduction of rotary kilns into new cement plants is universal, while many of the older mills are substituting them for their stationary kilns. Where rock, or rock and clay, form the raw materials, they are mixed and ground, and introduced into the rotary in the form of a dry powder. If marl or chalk furnish the carbonate of lime, the wet process of mixing and grinding is usually employed, as described on page 539, although in a few plants each of these materials is dried when entering the mill, and the operations are similar to those described below for rock mixtures, except that driers and disintegrators are substituted for stone crushers.

The process of manufacturing Portland cement from rock, or rock and clay mixtures, in plants equipped with rotary kilns, consists essentially of crushing the materials, — either separately or after mixing them, — drying, grinding, calcining in the rotaries, cooling, grinding to powder, and packing.

The details of the process will be best understood by briefly describing

the typical machinery shown in the illustrations. Various types and makes of grinding machinery will produce similar results, those selected being merely representative.

If two stones of fairly similar texture and each of uniform composition form the raw materials, they may be carefully weighed and thrown together into the breaker. Otherwise, they are treated separately, and mixed just before the grinding which precedes the calcination. A common type of breaker is the gyratory crusher shown in Fig. 93 on page 338, No. 5 or No. 6 being the usual size employed. This reduces the stone to a size varying from dust to about 2½-inch diameter. A further reduction in size to about ½-inch is accomplished in plants of modern design by crackers of the coffee mill type (see Fig. 165), or similar machinery.

Clay, if used, is dried in broken lumps, and then may be pulverized by passing it through a disintegrator consisting of two horizontal rolls, one corrugated or toothed and the other smooth.

An economical form of dryer for clay or stone consists of a long revolving steel tube about 4 feet in diameter, provided with shelves on its interior surface, formed by horizontal Z-bars. The hot gases from the kiln may be made to pass through the tube and meet the raw material.

By treating the two materials separately up to this point, an extremely accurate mixture is obtained by weighing the ingredients in a pair of automatic weighing machines (see Fig. 166), so arranged that one of the pair will not dump until both are charged.

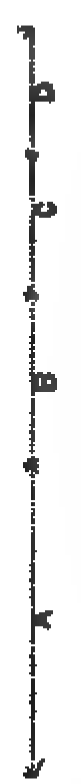

FIG. 165.—Coffee Mill Cracker. (*See p.* 532.)

Samples of the two materials are taken, just before mixing, at definite periods throughout the day, and analyzed to determine the correct proportions. A partial analysis showing the quantities of the principal constituents may be all that is necessary except at occasional intervals. The maintaining of correct proportions is one of the most essential elements in the manufacture.

Another grinding of the mixed materials in tube mills, Kent Mills, Griffin Mills (see pages 536 and 537), or similar machines, to a fineness which will pass a screen having 20 to 30 meshes per linear inch, completes the preparation for the rotary kilns. The actual fineness of the

ground stone at this point is such that 90% to 95% or even a higher percentage will pass a screen having 100 meshes to the linear inch. Fine grinding before burning is one of the secrets of successful manufacture.

The best type of rotary kiln (see Fig. 167) used for calcining dry materials, consists of an inclined steel tube from 60 to 150 feet long. The diameter is generally 6 or 7 feet, though often it is smaller than this at the upper end and then tapers to the larger size at a point about one-third of its length from the upper end. The lining may be of U-shaped fire-brick in order to present, as a non-conductor of heat, a hollow surface against the shell of the rotary. The lower end of the rotary is closed by a stationary brick wall, and through the center of this passes a pipe which feeds the petroleum, or more frequently the powdered coal which in a separate building is crushed to pea size and is generally pulverized in tube mills, or other pulverizing machines, so that about 90% passes a 100-mesh screen.

FIG. 166.—Tandem Automatic Weighing Machine. (*See p.* 532.)

The ground stone may be fed into the upper end of the rotary by a spiral conveyor enclosed in a pipe which is water-jacketed so that the material will not cake. The degree of calcination is governed by the supply of raw material, the speed of rotation of the rotary, which rests on rollers geared to a speed-changing device, and the quantity of fuel. If coal is used for fuel, it is fed by a blast from a fan, and the quantity is regulated by a spiral

FIG. 167.—Rotary Kiln. (*See p.* 533.)

conveyor running at changeable speed. The heat in the kiln is so intense that the coal burns as a gas without apparent smoke or cinder. The proper temperature, which is said to be 2700° to 3000° Fahr., is determined by the appearance of the burning stone. At a certain point in its descent the material becomes semi-vitrified and forms into irregular balls or clinkers, which roll around and finally fall out red-hot at the lower end in particles, most of which range in size from sand to 1-inch diameter. The clinker, when properly burned, is of a greenish black color with a faint glisten, and contains but few large pieces. It slightly resembles in appearance the clinker often found among the ashes of hard coal.

The output of a rotary kiln running on dry raw materials is nominally 150 to 200 barrels of finished cement in twenty-four hours. This is sometimes exceeded by as much as 25%.

The clinker, after being cooled in some form of cooler, is crushed by passing between horizontal rolls

or through some other form of crusher, and is then ready for the fine grinding, or, if desired, it may be stored either out of doors or under cover until needed. Strangely enough, wetting the cinder does not injure it provided it is dry when it enters the fine grinders.

The fine grinding is generally accomplished by passing the clinker through ball mills and then through tube mills, or by a single operation in such machines as the Griffin mill or the Kent mill. A section of a ball mill is shown in Fig. 168. It consists essentially of a cylindrical drum, lined with castings of hard, tough steel, and containing forged steel balls 8 or 10 inches in diameter. Rotation of the drum grinds the stone or clinker between the balls and the plates, and the powder passes through sections of screens — which for clinker have usually 20 to 28 meshes to the linear inch — into the hopper below. A single ball mill, such as is shown in sketch, running on clinker, should give an output of, say, 5 500 to 7 500 pounds per hour.

FIG. 168.—Ball Mill. (*See p.* 535.)

A tube mill (see Fig. 169) consists of a long horizontal cylinder filled

nearly to its axle with flint pebbles imported from Europe, which average about 2 to 3 inches in diameter. The cement is ground by rolling around with the flints. It is then thrown by centrifugal force against the screen, which regulates the fineness of grinding and prevents the passing of pieces of flint. A tube mill which passes, say, 250 barrels of cement per day, will require the renewal of the flint pebbles at the rate of about 600 lb. per week. More tube mills than ball mills, usually twice as many, are required for the finish grinding.

FIG. 169.—Tube Mill. (*See p.* 535.)

The Griffin mill (see Fig. 170) is used by many manufacturers in preference to ball and tube mills. The mill is driven by a horizontal pulley, from the center of which, by a universal joint, is suspended a vertical shaft having fixed at its lower extremity a crushing roll, which revolves on its axis at a speed of about 200 revolutions per minute, and also rotates by centrifugal force against the ring or die where the pulverizing is accomplished. The material to be ground passes first into the pan below the crushing roll, upon the under side of which are shoes or plows which stir it up and force it up between the roll and the die.

FIG. 170.—Griffin Mill. (*See p.* 536.)

The cement or stone is so finely powdered that, held in suspension by the moving air, it passes through a cylindrical screen above the roll, and falls through slots in the circumference of the pan into the hopper below, to be carried off by a conveyor. The screen in mills for grinding clinker is 30 to 32 mesh to the linear inch, but as it is placed vertically, it lets through only cement of such fineness that 75 to 80% of it will pass a 200-mesh sieve.

The Kent pulverizer, shown in Fig. 171, which is used in a few plants, consists essentially of an upright circular case containing within it three rolls surrounded by a revolving ring. The material is ground by passing between the internal circumference of this ring and the rolls, which are pressed against it by springs.

FIG. 171.—Kent Mill (*See p.* 537.)

It is customary to store the cement in bulk and weigh it out into bags or barrels as required for shipment. An automatic weighing machine similar to that shown in Fig. 166, page 533 (except that it is single instead of double), is a convenient apparatus for bagging. With this machine a weighing gang consists of three men. The nominal capacity of a single machine is 3 000 bags in ten hours, and the authors have known as many as 3 900 bags to be filled in this time.

In outlining the cement machinery, no reference has been made to the methods for conveying the material from one machine to another. Bucket

conveyors, belts and spiral conveyors are all more or less used. A spiral conveyor is a helical blade on a revolving shaft, set in a square or circular trough or tube of larger size than the spiral, so that the material packs around the circumference, and the blade comes in contact only with the powdered material.

Plaster of Paris (calcium sulphate $CaSO_4$), or gypsum ($CaSO_4 + 2H_2O$) the same substance in crystalline form, is an important addition to cement as a regulator of its setting, and from 1 to 2% is used in nearly all Portland cement manufactories. The gypsum must be added after the calcination and before the final grinding, in order to insure the proper result.

The laboratory of a cement plant is an important feature. Not only must the chemical composition of the raw materials and the finished product be analyzed (see Appendix I) at frequent periods, but the cement must be mechanically tested for fineness, time of setting, tensile strength at seven and twenty-eight days, and, perhaps most important of all, for soundness. Most manufacturers use some form of the accelerated or hot test. This is not only due to the fact that many engineers require the cement to pass an accelerated test for reception, but because the chemists in the cement factories consider this test of great value in checking up the quality of cement.

Wet Process with Rotary Kilns. The rotary or Ransome kiln was first used in England on wet materials. Rotaries have been widely, in fact almost universally, adopted in the United States for calcining dry materials, and more recently this field has been extended to use with slurry containing as much as 40% of water, which is pumped into the end of the rotary and dried by the same flame used for calcination. With kilns of ordinary length, Mr. Henry S. Spackman states* that at least 25% more fuel is required for burning than with dry materials, and the temperature of the gases in the chimney is about 400° Fahr., one-third to one-half that from dry kilns. The product per kiln, according to Mr. Spackman, is not much more than 100 barrels per kiln, or about one-half the output with dry materials.

Higher production than this has been attained by lengthening the kilns so as to utilize more thoroughly the heat of the flame. Lengths of 70 to 100 feet are used, or a cylindrical kiln about 60 feet in length and 6 feet in diameter, lined with firebrick, is connected at its upper end with an independent drying tube 40 to 50 feet long of slightly smaller diameter and with no lining. A kiln 6 feet in diameter by 60 feet long, with a 54-inch by 50-foot dryer extension, working on wet materials, has been known in certain cases to give an average capacity of from 135 to 140 barrels per day.†

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*Proceedings Philadelphia Engineers' Club, April, 1903.

†Statement of Allis-Chalmers Co. to the authors.

In the United States the raw materials most commonly employed in the wet process are marl and clay. The marl as it comes to the mill is broken up in some form of a disintegrator. The clay is dried and pulverized and is then mixed with the marl, which is about of the consistency of thick cream, in a pug mill or edge-runner. (See Fig. 172.)

Fig. 172.—Pug Mill. (*See p.* 539.)

In some cases the clay is ground and water is added to it before mixing with the marl.

The mixed materials must now be ground wet before burning. This is often accomplished in mill stones, consisting of a pair of horizontal

stones the upper one of which revolves upon an upright shaft, or in wet tube mills closely similar to that shown in Fig. 169 on page 536.

From the mills, it may be run into tanks, where it is sampled and its chemical composition exactly determined, and from there pumped into the ends of rotary kilns, which, as stated above, are usually made longer than those used in the dry process.

Centrifugal pumps may be employed for conveying the wet material, or if it is too thick for these to handle, plunger pumps may be resorted to. A more recent system of handling is by compressed air.

After calcination the treatment is similar to that in mills where dry raw materials are used.

Stationary Kilns. Before the introduction of rotary or revolving kilns all cement was burned in stationary kilns. Stationary kilns are of two general types: (1) intermittent kilns, which are completely charged and then burned, and (2) continuous kilns, where the fire is maintained continuously and the exhaust heat used to dry and heat the raw materials before burning them.

The most common form of intermittent kiln is the *Dome* or *Bottle Kiln*. This consists of a single shaft into which alternate layers of moist bricks of cement slurry and coke are placed by hand and burned. After cooling, the clinker is drawn out by hand through a door at the bottom, picked over to remove under-burned clinker, — which is of a yellowish shade instead of black, — and clinker which has fused to fragments of the firebrick lining.

The *Johnson Kiln* is a more economical form of intermittent kiln. The slurry is placed in chambers, and dried by the exhaust gases from the burning of the previous charge before being placed in the kilns.

Of the continuous kilns, the *Hoffman Ring Kiln* consists of several chambers or furnaces around a central chimney. As the material in one furnace is burned, the heat passes around through the other furnaces so as to raise the temperature of the bricks in them and utilize the exhaust heat.

In the *Schoefer Kiln*, which is also of the continuous type, the bricks and fuel are loaded from time to time into the upper end of the shaft, and pass down, increasing in temperature, through the flame, where the area is contracted, to be cooled below and drawn out at the bottom.

The *Dietzsch Kiln* is of a somewhat similar type of construction, except that hand-labor is required in passing the dried material into the heating chamber.

Comparison of Rotary and Stationary Kilns. Mr. Frederick H. Lewis* compares the three classes of kilns as follows:

Quantity of Fuel

Intermittent kilns	15 to 30 bbls. per day
Continuous shaft kilns	40 to 80 bbls. per day
Rotary kilns	120 to 250 bbls. per day

Fuel in Terms of Clinker Produced

Intermittent kilns require	25 to 35% of fuel (coke)
Continuous shaft kilns require	12 to 16% of fuel (coal)
Rotary kilns require	24 to 40% of fuel (coal)

The chief difference in cost between rotary and stationary kilns is for labor. In a rotary plant one sees the machinery running with only an occasional attendant, as no handling of the materials is required from the time they enter the mill until the cement is packed in bags or barrels for shipment. In the stationary kiln plant, even if brick machines are used for molding the slurry, a great deal of hand labor is required, as the kilns must be loaded and emptied by hand. Mr. Lewis estimates the labor cost with continuous kilns to range from three to five times the cost with rotaries.

NATURAL CEMENT MANUFACTURE

The process of manufacture of Natural cement consists, in brief, of burning a natural argillaceous limestone at low heat and grinding it to powder. The stone used in England is very soft, in fact nearly as disintegrated as marl.

Raw Material. Many of the limestones used for Natural cement contain a high proportion of magnesia and an excess of clay, while others are nearly free from magnesia. It must be calcined at a temperature much below that required for Portland cement or it will fuse to a slag which after grinding has no hydraulic properties. Suitable formations occur in many parts of the United States, one of the most noted being that found in the region of eastern New York where Rosendale cements are made. Sometimes the stone is taken entirely from one ledge, while in other cases mixtures of two strata are employed. Little attention is paid to the analysis of the rock, as there is a wide range in the required chemical composition of the product (see p. 47), and the price at which Natural cement is sold does not warrant great refinement.

Process of Manufacture of Natural Cement. There is less variety in the methods employed for producing Natural cement than for Portland.

**Engineering Record*, Dec. 17, 1898, p. 47, and personal correspondence.

In a typical plant, the stones, of about the size that would be required for a large crusher, are brought from the quarry in carts or cars and dumped directly into the top of the kilns, which are of boiler iron lined with firebrick. They have no chimneys, but are open at the top and of the same size throughout. Thick layers of stone are alternated with thin layers of pea coal. The clinker is drawn out at the bottom as it is burned.

In the older plants the burned clinker is crushed and then ground between mill stones, while the newer mills use grinding machinery similar to that in Portland cement plants. When burnt, Natural cement rock is more readily powdered than Portland cement clinker.

PUZZOLAN CEMENT MANUFACTURE*

Puzzolan cement is made in the United States from blast furnace slag mixed with slaked lime. In Europe, natural puzzolanic materials have been employed.

The process of manufacture consists essentially of cooling the slag, mixing it with slaked lime, and grinding to a very fine powder.

Slag for Puzzolan Cement. For making pig iron a blast furnace is charged with a mixture of iron ores, fluxes (consisting of limestone, either calcite or dolomite) and fuel, in the proper chemical proportions to produce, after reduction by heat, products of definite chemical composition. These resulting products are pig iron and slag. Any one unacquainted with metallurgy naturally thinks of blast furnace slag as a compound of iron. This is incorrect, as iron forms only a very small impurity.

All slags are not suitable for Puzzolan cement, as they ordinarily contain too high a percentage of magnesia and are often too high in alumina. The specifications for slag used in the manufacture of Steel Portland cement are as follows:†

Slag must analyze within the following limits :

	Per cent.
Silica plus alumina, not over	49
Alumina	13 to 16
Magnesia, under	4

Slag must be made in a hot furnace and must be of light gray color.

Slag must be thoroughly disintegrated by the action of a large stream of cold water directed against it with considerable force. This contact should be made as near the furnace as is possible."

Mr. E. Candlot says‡ "The slag must be basic; according to Mr. Tet-

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*An investigation of the manufacture and properties of Puzzolan cement is given in Report of Board of Engineers, U. S. A., 1900, on Steel Portland cement.

†Report of Board of Engineers, U. S. A., 1900, on Steel Portland Cement.

‡Ciments et Chaux Hydrauliques, 1898, p. 157.

majer, when the ratio $\frac{CaO}{SiO_2}$ falls below unity the slag is useless; the ratio of silica to alumina must be between 0.45 and 0.50. According to Mr. Prost, the composition of slags habitually used in the manufacture of Puzzolan cements must be nearly represented by the formula 2 SiO_2, Al_2O_3, 3 CaO."

Mr. E. C. Eckel* gives the following analyses of slag and slag cement:

Analyses of Slags in Actual Use and Composition of Slag Cements

	SLAG			CEMENT		
CONSTITUENT.	Choindez, Switzerland.	Saulnes, France.	Chicago, Ill.	Choindez, Switzerland.	Saulnes, France.	Chicago, Ill.
SiO_2	26.24	31.50	32.20	19.5	22.45	28.95
Al_2O_3	24.74	16.62	15.50	17.5	13.95	11.40
FeO	0.49	0.62			3.30	0.54
CaO	46.83	46.10	48.14	54.0	51.10	50.29
MgO	0.88		2.27		1.35	2.96
CaS	0.59					
$CaSO_4$	0.32					
S						1.37
SO_3					0.35	
Loss on ignition					7.50	3.39
$\frac{CaO}{SiO_2}$	1.78	1.46	1.49			
$\frac{Al_2O_3}{SiO_2}$	0.93	0.52	0.48			

Process of Manufacture of Puzzolan Cement. No kilns are required except for burning the lime. Molten slag as it flows from the blast furnace is granulated by coming in contact with a stream of cold water. This renders the product more strongly hydraulic, and most of the sulphur is removed as it strikes the water. As sent to the cement plant, it usually contains from 30% to 40% of water, and the first operation is to pass it through a dryer. The dried slag may or may not have a preliminary grinding before adding the slaked lime.

The lime is produced by burning a pure limestone, and then slaking it with water to which has been added a small percentage of caustic soda or other similar material, to make the resulting cement quicker setting. After drying, the slaked lime is mixed with the slag and ground in ball mills and tube mills, or in other forms of fine grinding machinery, and is ready for packing in bags or barrels for shipment.

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*Mineral Resources of the United States, 1901.

CHAPTER XXIX

REFERENCES TO CONCRETE LITERATURE

While this chapter is not a complete bibliography of concrete literature, it presents a comprehensive list of valuable books and articles relating to the subject.

Under General References the names of authors are arranged alphabetically. The various subject headings under Subject References are also arranged alphabetically, and the references are printed in order of dates, the latest first. Articles are usually described by their subject-matter instead of giving their titles verbatim. In the case of similar articles printed in two or more periodicals, preference is generally given to the one bearing the earlier date. For references to this treatise see the Index.

ABBREVIATIONS

The following abbreviations (most of which correspond to those adopted by the Engineering Index) are employed:

Arch. Rec. — Architectural Record. New York.
Can. Eng. — Canadian Engineer. Montreal, Can.
Comptes Rendus. — Comptes Rendus de l'Académie des Sciences. Paris.
Deutsche Bau. — Deutsche Bauzeitung. Berlin.
Eng. Mag. — Engineering Magazine. New York & London.
Eng. News. — Engineering News. New York.
Eng. Rec. — Engineering Record. New York.
Gen. Civ. — Génie Civil. Paris.
Ins. Eng. — Insurance Engineering. Boston.
Int. Eng. Cong. — International Engineering Congress, St. Louis, 1904.
Jour. Am. Chem. Soc. — Journal American Chemical Society. Easton, Pa.
Jour. Assn. Eng. Socs. — Journal of the Association of Engineering Societies, Philadelphia.
Jour. Fr. Inst. — Journal Franklin Institute. Philadelphia.
Jour. W. Soc. Engs. — Journal of the Western Society of Engineers, Chicago.
Munic. Engng. — Municipal Engineering. Indianapolis.
Oest. Monatschr. f. d. Oeff. Baudienst. — Oesterreichische Monatsschrift für den Oeffentlichen Baudienst. Vienna.
Pro. Am. Soc. Civ. Engs. — Proceedings of the American Society of Civil Engineers. New York.
Pro. Am. Soc. Test. Mat. — Proceedings of American Society for Testing Materials. Philadelphia.

Pro. Assn. Ry. Supts. — Proceedings of the American Association of Railway Superintendents of Bridges and Buildings. New York.
Pro. Engs. Club of Phila. — Proceedings Engineers' Club. Philadelphia.
Pro. Engs. Soc. of W. Penn. — Proceedings of Engineers' Society of Western Pennsylvania. Pittsburgh.
Pro. Inst. Civ. Engs. — Proceedings of the Institution of Civil Engineers. London.
Ry. & Eng. Rev. — Railway & Engineering Review. Chicago.
R. R. Gaz. — Railroad Gazette. New York.
Rept. Chief of Engs., U. S. A. — Report of Chief of Engineers, U. S. A.
Rept. Eng. Dept. — Report of Engineering Department, Washington, D. C.
Rept. Met. W. & S. Board. — Report of Metropolitan Water & Sewerage Board, Massachusetts.
Revue Gen. des Chemins de Fer. — Revue Générale des Chemins de Fer. Paris.
Rev. Tech. — Revue Technique. — Paris.
Schw. Bauz. — Schweizerische Bauzeitung. Zürich.
Tech. — Technograph. University of Illinois. Champaign, Ill.
Tech. Qr. — Technology Quarterly. Boston.
Trans. Am. Soc. Civ. Engs. — Transactions American Society of Civil Engineers. New York.
Trans. Am. Soc. Mech. Engs. — Transactions American Society of Mechanical Engineers. New York.

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Location.	Max. span ft.	Max. rise ft.	Crown thickness ft.	Reinforcement.	Authority.
Paterson, N. J.,	54	2.5	1.8	11 ribs about 4 ft. apart	*Eng. Rec.* Sept., 1904, p. 303
Plainwell, Mich.,	54	8	1.25	4-inch 6-lb. channels 1.9 ft. apart	*Eng. News*, May, 1904, p. 456
Waterloo, Iowa,	72	7.2	1.18	Steel ribs	*Eng. Rec.*, Feb., 1904, p. 185
Yellowstone River,	120	15	2.0	Lattice girders	*Eng. News*, Jan., 1904, p. 25

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* An asterisk precedes the references which are especially noteworthy.

Location.	Max. span ft.	Max. rise ft.	Crown thickness ft.	Reinforcement.	Authority
Plano, Ill.,	75	38½	3	¾" and ⅞" corrugated bars	*Eng. Rec.*, Jan., 1904, p. 18
3rd St., Dayton, Ohio,	110	14.25	2.1	Melan, 4 angles, latticed	Edwin Thacher, 1904
Newark, N. J.,	132	16.2	3	Melan, 4 angles, latticed	Edwin Thacher, 1904
Kankakee, Ill.,	73	8.4	1.33	Thacher, rods near top and bottom	Edwin Thacher, 1904
Mishawaka, Ind.,	110	14	2	Melan, 4 angles, latticed	Edwin Thacher, 1903
Prospect Ave., N. Y,	85	8½	2.25	Corrugated bars	*Eng. News*, Dec., 1903, p. 588
Riverside, Cal.,	87	36.9	3.5	None	*Eng. News*, Oct., 1903, p. 353
Leominster, Mass.,	45	6.25	1.1	Round rods anchored	J. R. Worcester, 1903
Des Moines River,	100	28	1.67	Melan	*Cement*, July, 1902, p. 200
Zanesville, Ohio,	122	11.5	2.5	¾" x 5" bars	*Eng. News*, March, 1902, p. 261
Concord, Mass.,	66	7	1.1	None	J. R. Worcester, 1901
Lansing, Mich.,	120	23	2	Melan, 4 angles, latticed	Edwin Thacher, 1901
South Bend, Ind.,	100	11	2.5	Melan, 4 angles, latticed	Edwin Thacher
Chatellerault, France,	164	15.7	1.7	Hennebique	Revue Gen. des Chemins de Fer, Dec., 1901
Kirchheim, Germany,	124.6	18	2.6	None	*Eng. News*, Oct., 1899, p. 246
Germany,	132	14.7	0.82	Monier	*Eng. News*, Sept., 1899, p. 179
Switzerland,	128	11	0.56	Monier	*Eng. News*, Sept., 1899, p. 179
Southern Ry., Austria,	32.8	3.3	0.5	Monier	*Eng. News*, Sept., 1899, p. 179
Topeka, Kan.,	125	12	1.8	Melan beams	*Eng. Rec.*, April 16, 1898
Inzigkofen, Germany,	140	14.5	2.3	33 000 lb. cast iron	*Eng. News*, Sept., 1896, p. 178
Munderkingen, Germany,	164	16.4	3.3	None	Inst. Civ. Engs., V. 119, p. 224
Cincinnati, Ohio,	70	10	1.25	Melan beams	*Eng. News*, Oct., 1895, p. 214
Maryborough, Queensl'd	50	4	1.25	Steel rails	*Engng.*, London, May 10, 1895, p 395
Neuhäusel, Hungary,	55.78	3.7	0.82	Skeleton girders	Inst. Civ. Engs., V., 114, p. 402
Philadelphia, Penn.,	25.39	6.5	3	1½" mesh, ¼" wire netting	*Eng. News*, Sept., 1893, p. 189

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APPENDIX I

METHOD SUGGESTED FOR THE ANALYSIS OF LIMESTONES, RAW MIXTURES, AND PORTLAND CEMENTS BY THE COMMITTEE ON UNIFORMITY IN TECHNICAL ANALYSIS OF THE AMERICAN CHEMICAL SOCIETY, WITH THE ADVICE OF W. F. HILLEBRAND.

Solution: One-half gram of the finely powdered substance is to be weighed out and, if a limestone or unburned mixture, strongly ignited in a covered platinum crucible over a strong blast for 15 minutes, or longer if the blast is not powerful enough to effect complete conversion to a cement in this time. It is then transferred to an evaporating dish, preferably of platinum for the sake of celerity in evaporation, moistened with enough water to prevent lumping, and 5 to 10 c. c. of strong HCl added and digested, with the aid of gentle heat and agitation, until solution is completed. Solution may be aided by light pressure with the flattened end of a glass rod.* The solution is then evaporated to dryness, as far as this may be possible on the steam bath.

Silica: The residue, without further heating, is treated at first with 5 to 10 c. c. of strong HCl which is then diluted to half strength or less, or upon the residue may be poured at once a larger volume of acid of half strength. The dish is then covered and digestion allowed to go on for 10 minutes on the bath, after which the solution is filtered and the separated silica washed thoroughly with water. The filtrate is again evaporated to dryness, the residue, without further heating, taken up with acid and water and the small amount of silica it contains separated on another filter paper. The papers containing the residue are transferred wet to a weighed platinum crucible, dried, ignited, first over a Bunsen burner until the carbon of the filter is completely consumed, and finally over the blast for 1 minute and checked by a further blasting for 10 minutes or to constant weight. The silica, if great accuracy is desired, is treated in the crucible with about 10 c. c. of HFl and four drops of H_2SO_4 and evaporated over a low flame to complete dryness. The small residue is finally blasted, for a minute or two, cooled and weighed. The difference

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*If anything remains undecomposed it should be separated, fused with a little Na_2CO_3, dissolved and added to the original solution. Of course a small amount of separated non-gelatinous silica is not to be mistaken for undecomposed matter.

between this weight and the weight previously obtained gives the amount of silica.[*]

Al_2O_3 and Fe_2O_3: The filtrate, about 250 c.c., from the second evaporation for SiO_2, is made alkaline with NH_4OH after adding HCl, if need be, to insure a total of 10 to 15 c.c. strong acid, and boiled to expel excess of NH_3, or until there is but a faint odor of it, and the precipitated iron and aluminum hydrates, after settling, are washed once by decantation and slightly on the filter. Setting aside the filtrate, the precipitate is dissolved in hot dilute HCl, the solution passing into the beaker in which the precipitation was made. The aluminum and iron are then re-precipitated by NH_4OH boiled, and the second precipitate collected and washed on the same filter used in the first instance. The filter paper, with the precipitate, is then placed in a weighed platinum crucible, the paper burned off and the precipitate ignited and finally blasted 5 minutes, with care to prevent reduction, cooled and weighed as $Al_2O_3 + Fe_2O_3$.[†]

Fe_2O_3: The combined iron and aluminum oxides are fused in a platinum crucible at a very low temperature with about 3 to 4 grams of $KHSO_4$, or, better, $NaHSO_4$, the melt taken up with so much dilute H_2SO_4 that there shall be no less than 5 grams absolute acid and enough water to effect solution on heating. The solution is then evaporated and eventually heated till acid fumes come off copiously. After cooling and redissolving in water the small amount of silica is filtered out, weighed, and corrected by HFl and H_2SO_4.[‡] The filtrate is reduced by zinc, or preferably by hydrogen sulphide, boiling out the excess of the latter afterwards while passing CO_2 through the flask, and titrated with permanganate.[§] The strength of the permanganate solution should not be greater than .0040 gr. Fe_2O_3 per c.c.

CaO: To the combined filtrate from the $Al_2O_3 + Fe_2O_3$ precipitate a few drops of NH_4OH are added, and the solution brought to boiling. To the boiling solution 20 c.c. of a saturated solution of ammonium oxalate is added, and the boiling continued until the precipitated CaC_2O_4 assumes a well-defined granular form. It is then allowed to stand for 20 minutes,

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[*] For ordinary control work in the plant laboratory this correction may, perhaps, be neglected, the double evaporation never.

[†] This precipitate contains TiO_2, P_2O_5, Mn_3O_4.

[‡] This correction of Al_2O_3 Fe_2O_3 for silica should not be made when the HFl correction of the main silica has been omitted, unless that silica was obtained by only one evaporation and filtration. After two evaporations and filtrations 1 to 2 mg. of SiO_2 are still to be found with the Al_2O_3 Fe_2O_3.

[§] In this way only is the influence of titanium to be avoided and a correct result obtained for iron.

or until the precipitate has settled, and then filtered and washed. The precipitate and filter are placed wet in a platinum crucible, and the paper burned off over a small flame of a Bunsen burner. It is then ignited, redissolved in HCl, and the solution made up to 100 c.c. with water. Ammonia is added in slight excess, and the liquid is boiled. If a small amount of Al_2O_3 separates, this is filtered out, weighed, and the amount added to that found in the first determination, when greater accuracy is desired. The lime is then re-precipitated by ammonium oxalate, allowed to stand until settled, filtered, and washed,* weighed as oxide by ignition and blasting in a covered crucible to constant weight, or determined with dilute standard permanganate.†

MgO: The combined filtrates from the calcium precipitates are acidified with HCl, and concentrated on the steam bath to about 150 c.c., 10 c.c. of saturated solution of Na $(NH_4)HPO_4$ are added, and the solution boiled for several minutes. It is then removed from the flame and cooled by placing the beaker in ice water. After cooling, NH_4OH is added drop by drop with constant stirring until the crystalline ammonium-magnesium ortho-phosphate begins to form, and then in moderate excess, the stirring being continued for several minutes. It is then set aside for several hours in a cool atmosphere and filtered. The precipitate is redissolved in hot dilute HCl, the solution made up to about 100 c.c., 1 c.c. of a saturated solution of $Na(NH_4)HPO_4$ added, and ammonia drop by drop, with constant stirring until the precipitate is again formed as described and the ammonia is in moderate excess. It is then allowed to stand for about 2 hours when it is filtered on a paper or a Gooch crucible, ignited, cooled and weighed as $Mg_2P_2O_7$.

K_2O and Na_2O: For the determination of the alkalies, the well-known method of Prof. J. Lawrence Smith is to be followed, either with or without the addition of $CaCO_3$ with NH_4Cl.

SO_3: One gram of the substance is dissolved in 15 c.c. of HCl, filtered and residue washed thoroughly.‡

The solution is made up to 250 c.c. in a beaker and boiled. To the boiling solution 10 c.c. of a saturated solution of $BaCl_2$ is added slowly drop by drop from a pipette and the boiling continued until the precipitate is well formed, or digestion on the steam bath may be substituted for the

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*The volume of wash water should not be too large. *Vide* Hillebrand.

†The accuracy of this method admits of criticism, but its convenience and rapidity demand its insertion.

‡Evaporation to dryness is unnecessary, unless gelatinous silica should have separated and should never be performed on a bath heated by gas. *Vide* Hillebrand.

boiling. It is then set aside over night, or for a few hours, filtered, ignited, and weighed as $BaSO_4$.

Total Sulphur: One gram of the material is weighed out in a large platinum crucible and fused with Na_2CO_3 and a little KNO_3, being careful to avoid contamination from sulphur in the gases from source of heat. This may be done by fitting the crucible in a hole in an asbestos board. The melt is treated in the crucible with boiling water and the liquid poured into a tall, narrow beaker and more hot water added until the mass is disintegrated. The solution is then filtered. The filtrate contained in a No. 4 beaker is to be acidulated with HCl and made up to 250 c.c. with distilled water, boiled, the sulphur precipitated as $BaSO_4$ and allowed to stand over night or for a few hours.

Loss on Ignition: Half a gram of cement is to be weighed out in a platinum crucible, placed in a hole in an asbestos board so that about $\frac{3}{5}$ of the crucible projects below, and blasted 15 minutes, preferably with an inclined flame. The loss by weight, which is checked by a second blasting of 5 minutes, is the loss on ignition.

May, 1903:

Recent investigations have shown that large errors in results are often due to the use of impure distilled water and reagents. The analyst should, therefore, test his distilled water by evaporation and his reagents by apappropriate tests before proceeding with his work.

APPENDIX II

ADDITIONAL FORMULAS FOR REINFORCED CONCRETE BEAMS*

Practical working formulas suited to all ordinary cases of reinforced concrete design are presented in Chapter XIV. For reasons there given, the "straight line" theory — *i.e.*, the theory in which the modulus of elasticity of concrete in compression is assumed to be constant within usual working limits — is adopted as our standard and the concrete is assumed to bear no tension.

The various other rational formulas† which have been advanced by different mathematicians are based upon the same analytical methods of treatment, but on different assumptions of stress. Many have complicated their equations by taking moments about the neutral axis instead of about the centers of tension or compression, but the general principles of the deduction are the same, and in accordance with the analysis in Chapter XIV.

It is possible to evolve by calculus a general formula which satisfies all of the various hypotheses,‡ but the treatment is omitted and only the more practical demonstrations are given.

The chief points of the various hypotheses advanced which differ from the one which we have adopted in Chapter XIV are:

(1) Concrete below neutral axis bears tensile stress in proportion to the strain upon it.

(2) Concrete below neutral axis bears a small proportion of the tensile stress.

(3) Compressive stress in concrete varies as a curve instead of as a straight line, and the concrete bears no tension.

(4) Concrete carries both tensile and compressive stress, and these stresses both vary as curves.

Since the concrete actually does bear tension under light loading, the formulas with this assumption are given on page 565.

Formulas which assume the compressive stress to vary as a parabola are given on page 567.

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*The authors are indebted to Prof. Frank P. McKibben for the formulas in this Appendix, which have been especially prepared by him for this Treatise.

†See Christophe's Béton Armé and Morel's Ciments Armé, 1902.

‡See Burr's Materials of Engineering, 1903, p. 633.

With these formulas, and the complete analysis in Chapter XIV, formulas based on various other assumptions may be readily evolved.

In certain cases it is advisable to place steel in the compression portion of the beam, and formulas with this arrangement are therefore given on page 563.

Formulas for T-sections will be found on page 569.

Since the general method of derivation of all these formulas is similar to that of formulas in Chapter XIV, many of the intermediate steps are omitted in the demonstrations.

NOTATION

The same notation is adopted in this Appendix as in Chapter XIV, with the additional symbols necessary.

Let

h = height of beam.
h' = thickness of slab, *i.e.*, thickness of T-flange.
b = breadth of beam.
b' = breadth of T-flange.
p = ratio of cross-section of steel in tension to cross-section of beam above this steel.
p' = ratio of cross-section of steel in compression to cross-section of beam above the steel in tension.
C = unit compressive stress in outside fiber of concrete.
T = unit tensile stress, or pull, in outside fiber of concrete.
S = unit tensile stress, or pull, in steel.
S' = unit compressive stress in steel.
E_c = modulus of elasticity of concrete in compression.
E_t = modulus of elasticity of concrete in tension.
E_s = modulus of elasticity of steel.
$r = \frac{E_s}{E_c}$
x = ratio of depth of neutral axis to depth of steel in tension.
xd = distance from outside compressive surface to neutral axis in beam in which the depth to steel in tension is d.
e = extra thickness of concrete below steel in tension.
a = ratio of depth of steel in compression to depth of steel in tension.
M_R = moment of resistance.
M_B = bending moment.

STEEL IN TOP AND BOTTOM OF BEAM, NO TENSION IN CONCRETE

In certain structures, such as arch bridges, the distribution of the load may be such as to cause tension in the top of the member requiring the introduction of steel near its upper surface, and this steel affects the location of the neutral axis, and also increases the strength in the upper portion of the beam during normal loading, when the upper portion of the beam is in compression.

Formulas, adopting as usual the assumption of a constant modulus of elasticity and no tension in the concrete, in which the compressive stresses are partially borne by the steel in the upper portion of the beam, are as follows:

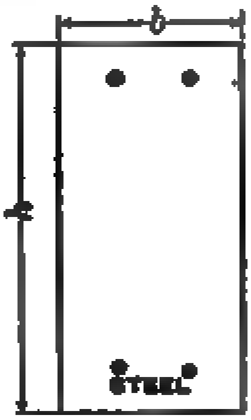

FIG. 173.—Resisting Forces with Steel in Top and Bottom of Beam. (*See p.* 563.)

Formulas. Deformations are assumed to vary directly as distance from neutral axis. (See p. 286.)

Hence from Fig. 173

$$\frac{\frac{S}{E_s}}{\frac{C}{E_c}} = \frac{d(1-x)}{xd} = \frac{1-x}{x}. \quad \text{Whence } x = \frac{1}{1+\frac{S}{Cr}} \qquad (1)$$

Also,
$$S' = S\frac{x-a}{1-x} \quad (2) \qquad \text{and } S' = Cr\frac{x-a}{x} \qquad (3)$$

$$S = Cr\frac{1-x}{x} \quad (4) \qquad \text{and } C = \frac{S}{r}\,\frac{x}{1-x} \qquad (5)$$

Equating the horizontal forces acting on the cross-section of the beam, we have:

$$bd\left(\frac{Cx}{2} + p'S'\right) = bdpS$$

Whence
$$p = \frac{1}{S}\left(\frac{Cx}{2} + p'S'\right) = \frac{1}{S}\left(\frac{S}{2r}\,\frac{x^2}{1-x} + p'S\frac{x-a}{1-x}\right)$$

Hence
$$p = \frac{x^2}{2r(1-x)} + p'\frac{x-a}{1-x} \qquad (6)$$

Solving equation (6) for x,

$$x = \sqrt{2r(p+p'a)+r^2(p+p')^2} - r(p+p') \qquad (7)$$

Taking moments about the center of pull in the steel, we have

$$M_R = \frac{bCxd}{2}\left(d - \frac{xd}{3}\right) + S'p'bd(d-ad)$$

$$M_R = bd^2\left[\frac{Cx}{2}\left(1-\frac{x}{3}\right) + S'p'(1-a)\right]$$

or by eliminating S' by means of equation (3),

$$M_R = Cbd^2\left[\frac{x}{2}\left(1-\frac{x}{3}\right) + \frac{rp'(x-a)(1-a)}{x}\right] \qquad (8)$$

Taking moments about the center of compression stress in the steel, we have

$$M_R = bd^2\left[Sp(1-a) - \frac{Cx}{2}\left(\frac{x}{3}-a\right)\right]$$

or by eliminating C,

$$M_R = Sbd^2\left[p(1-a) - \frac{x^2}{2r(1-x)}\left(\frac{x}{3}-a\right)\right] \qquad (9)$$

Then taking moments about center of compression in concrete:

$$M_R = bd^2\left[Sp\left(1-\frac{x}{3}\right) + S'p'\left(\frac{x}{3}-a\right)\right]$$

or by eliminating S,

$$M_R = S'bd^2\left[p\left(\frac{1-x}{x-a}\right)\left(1-\frac{x}{3}\right) + p'\left(\frac{x}{3}-a\right)\right] \qquad (10)$$

STEEL IN BOTTOM OF BEAM, CONCRETE BEARING TENSION

In the earlier stages of loading of reinforced concrete beams, the deformation curves (see Fig. 89, p. 288) indicate that the concrete actually bears a portion of the pull. Although it is not good practice to consider this pull in the design of beams, but, instead, it is customary to take the working strength as a factor of the ultimate, or nearly the ultimate, strength of the beam, the following formulas are useful for determining the actual stresses and for calculating deflections at the earliest stages of loading.

Formulas. Since elongation of steel and concrete at the same point must be equal, and since cross-sectional planes are assumed to remain plane

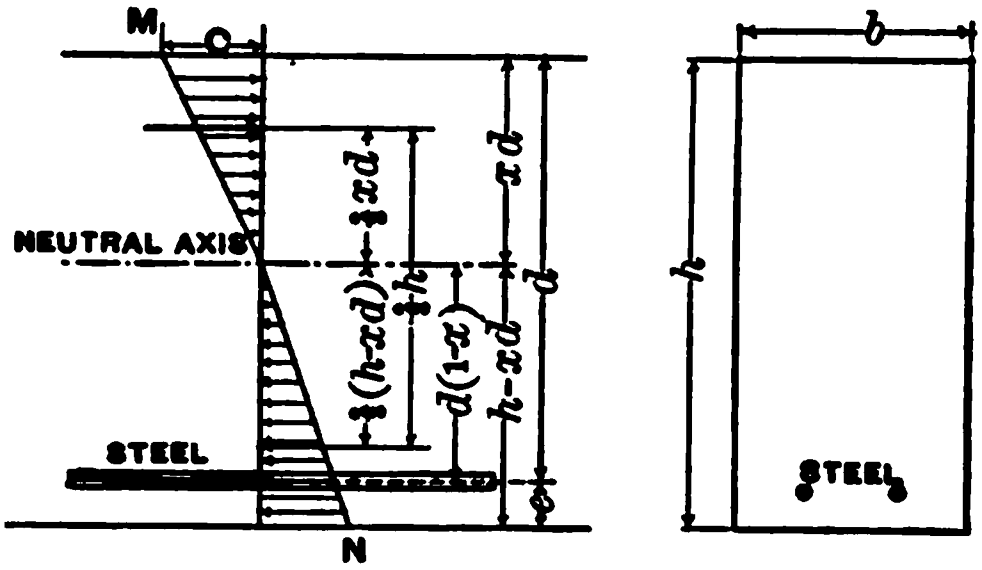

FIG. 174.—Resisting Force with Steel Bearing Tension. (*See p.* 565.)

during bending, we have from Fig. 174 the following equations:

$$\frac{\frac{S}{E_s}}{\frac{T}{E_t}} = \frac{d-xd}{h-xd} \text{ hence } S = \frac{E_s}{E_t} T \frac{d-xd}{h-xd} \tag{11}$$

$$C = \frac{E_c}{E_t} T \frac{xd}{h-xd} \tag{12}$$

$$S = \frac{E_s}{E_c} C \frac{1-x}{x} \quad (13) \quad \text{also } T = \frac{E_t}{E_c} C \frac{h-xd}{xd} \tag{14}$$

Equating horizontal forces on the section we have

$$\frac{bCxd}{2} = pSbd + \frac{Tb(h-xd)}{2} \tag{15}$$

The elimination of S and T from (15) gives

$$\frac{xd}{2} = pd \frac{E_s}{E_c} \frac{1-x}{x} + \frac{E_t}{E_c} \frac{(h-xd)^2}{2xd} \tag{16}$$

From which
$$p = \frac{1}{2(1-x)} \left[\frac{E_c}{E_s} x^2 - \frac{E_t}{E_s} \left(\frac{h-xd}{d} \right)^2 \right] \tag{17}$$

Solving equation (17) for x,

$$x = \sqrt{\frac{2p + \frac{E_t}{E_s} \frac{h^2}{d^2}}{\frac{E_c}{E_s} - \frac{E_t}{E_s}} + \left(\frac{p + \frac{E_t}{E_s} \frac{h}{d}}{\frac{E_c}{E_s} - \frac{E_t}{E_s}} \right)^2} - \frac{p + \frac{E_t}{E_s} \frac{h}{d}}{\frac{E_c}{E_s} - \frac{E_t}{E_s}} \tag{18}$$

Taking moments about the center of the pull in the concrete, the center of compression in the concrete and the center of pull in the steel respectively, we have the three following equations for the moment of resistance:

$$M_R = Spbd\left(d - \frac{xd}{3} - \frac{2h}{3}\right) + \frac{Cbxd}{2}\frac{2h}{3}$$

$$= Sbd\left[p\left(d - \frac{xd}{3} - \frac{2h}{3}\right) + \frac{E_c}{E_s}\frac{hx^2}{3(1-x)}\right] \tag{19}$$

or

$$M_R = Spbd\left(d - \frac{xd}{3}\right) + \frac{Tb(h - xd)}{2}\frac{2h}{3}$$

$$= Tb\left[pd^2\left(1 - \frac{x}{3}\right)\frac{E_s}{E_t}\frac{1-x}{h - xd} + \frac{h}{3}(h - xd)\right] \tag{20}$$

or

$$M_R = \frac{Cbxd}{2}\left(d - \frac{xd}{3}\right) - \frac{Tb(h - xd)}{2}\left(d - \frac{xd}{3} - \frac{2h}{3}\right)$$

$$= \frac{Cb}{2}\left[xd^2\left(1 - \frac{x}{3}\right) - \frac{E_t}{E_c}\frac{(h - xd)^2}{xd}\left(d - \frac{xd}{3} - \frac{2h}{3}\right)\right] \tag{21}$$

If now $E_t = E_c$, that is, if the modulus of elasticity of concrete is the same in tension as in compression, the line MN becomes straight.

Equation (17) then becomes, letting $\frac{E_s}{E_c} = r$,

$$p = \frac{1}{2}\frac{h}{rd^2}\left(\frac{2xd - h}{1 - x}\right) \tag{22}$$

From which

$$x = \frac{h^2 + 2prd^2}{2dh + 2prd^2} \tag{23}$$

Equation (19) is not changed.

Equation (20) simply has E_c instead of E_t.

Equation (21) becomes

$$M_R = \frac{Cb}{2}\left[xd^2\left(1 - \frac{x}{3}\right) - \frac{(h - xd)^2}{xd}\left(d - \frac{xd}{3} - \frac{2h}{3}\right)\right]$$

or

$$M_R = \frac{Cbh}{2}\left[2d - \frac{h}{x} - h + \frac{2h^2}{3xd}\right] \tag{24}$$

COMPRESSIVE STRESS AS A PARABOLA, STEEL IN BOTTOM OF BEAM, NO TENSION IN CONCRETE

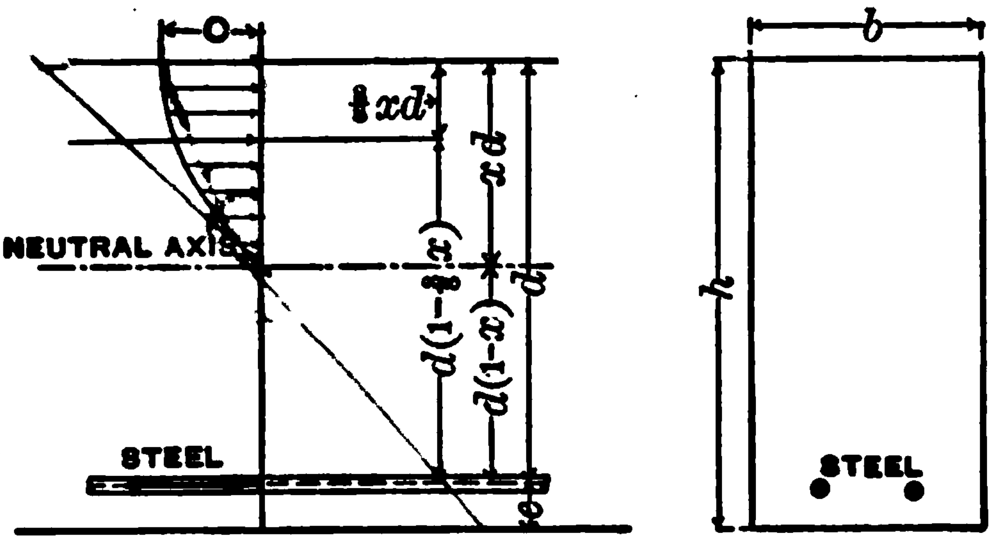

FIG. 175.—Resisting Forces with Pressure Varying as a Parabola. (*See p.* 567.)

Many experiments upon the compression of concrete show a gradually decreasing modulus of elasticity as the load increases. From the form of the stress-deformation curve of these specimens, the stress on the compression side of a beam is sometimes assumed to vary as a parabola instead of as a straight line. This method was first suggested in the United States by Prof. W. Kendrick Hatt.* The formulas which follow present this method of analysis, and permit the comparison† of results by this assumption, with results of the straight line theory adopted by the authors in Chapter XIV.

Formulas. As in preceding cases, from Fig. 175,

$$\frac{\frac{S}{E_s}}{\frac{C}{E_c}}=\frac{d(1-x)}{xd}=\frac{1-x}{x}$$

hence

$$x=\frac{1}{1+\frac{S}{Cr}} \tag{25}$$

From which

$$C=\frac{S}{r}\frac{x}{(1-x)} \tag{26}$$

Equating horizontal forces on the section of the beam we have

$$pbdS=\frac{2\,bCxd}{3}\text{, or more simply, } pS=\frac{2\,Cx}{3} \tag{27}$$

Substitute the value of x from (25) and we have:

$$p=\frac{2}{3\left(\frac{S}{C}\right)\left(1+\frac{S}{Cr}\right)} \qquad 3\left(\frac{f}{c}\right)\left(1+\frac{f}{nc}\right) \quad \frac{f}{c}(\tag{28}$$

---

*Proceedings American Society for Testing Materials, 1902.

†See p. 300 for comparative values by the two theories.

which gives the ratio of steel required for any consistent values of S, C, E_s, E_c. The position of the neutral axis is dependent upon the per cent of steel and the moduli of elasticity of steel and concrete, and the value of x may be found by substituting in (27) the value of S from equation (26) Thus

$$\frac{2Cx}{3} = pCr\frac{1-x}{x}, \text{ or } p = \frac{2}{3}\frac{x^2}{(1-x)r}$$

Solving this quadratic equation and using the positive sign after taking the square root,

$$x = \sqrt{\frac{3}{2}rp + \left(\frac{3}{4}rp\right)^2} - \frac{3}{4}rp$$

or in another form,

$$x = \frac{3}{4}rp\left[\sqrt{\frac{8}{3rp} + 1} - 1\right] \qquad (29)$$

The moment of resistance may be found by taking moments about the center of compression in the concrete, thus,

$$M_R = Spbd^2\left(1 - \frac{3}{8}x\right) \qquad (30)$$

or by taking moments about the center of pull in the steel,

$$M_R = \frac{2}{3}Cxbd^2\left(1 - \frac{3}{8}x\right) \qquad (31)$$

Eliminating x from these equations by substituting its value from equation (25), and also substituting the value of p from equation (28), we have

$$M_R = \frac{2}{3}Sbd^2\frac{1}{\left(\frac{S}{C}\right)\left(1 + \frac{S}{Cr}\right)}\left[1 - \frac{3}{8\left(1 + \frac{S}{Cr}\right)}\right] \qquad (32)$$

or

$$M_R = \frac{2}{3}Cbd^2\frac{1}{1 + \frac{S}{Cr}}\left[1 - \frac{3}{8\left(1 + \frac{S}{Cr}\right)}\right] \qquad (33)$$

T-SHAPED SECTION OF BEAM

When a reinforced concrete floor slab and beam are built as one piece, the slab undoubtedly adds strength to the beam. However, practical experiments upon this combination have been so few that beams are

generally designed, as described in Chapter XIV, without reference to this increase in strength.

The following formulas present the method of analysis which may be followed if the beam and the slab are assumed to act as one member.

The formulas are based upon the assumption that the intensity of the compression in the concrete does not diminish from the web outward towards the edges of the flange. For a section having a narrow flange, this would be practically correct, but with a wide flange, as in the case where the flange is a part of the floor slab, the intensity of the compression in the flange would diminish from the web outward. If this pressure is assumed to decrease, either uniformly or otherwise, the formulas may be modified accordingly.

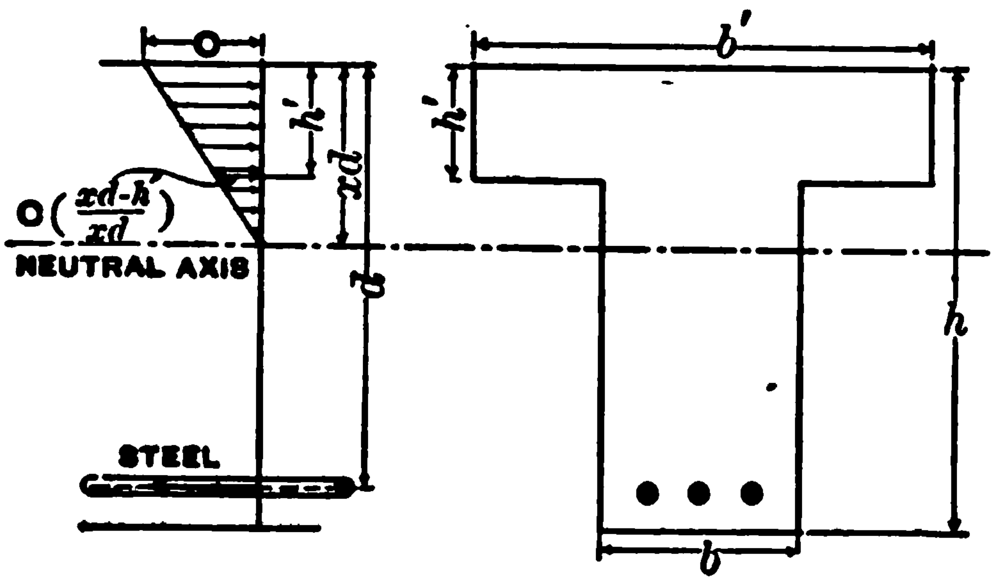

FIG. 176.—Resisting Forces in T-shaped Section of Beam. (*See p.* 569.)

Assuming the compression to be distributed as shown in the diagram, and the steel to take all the tension, the formulas given below may be deduced as in the preceding cases. The ratio of steel is taken as the ratio of the area of the steel to the area of the beam above the steel exclusive of the flange, that is, to the area *bd*, Fig. 176.

Case I. Neutral Axis Below Flange, $xd > h'$.

Neglect the slight amount of compression in the web below the intersection of the web and flange.

As in previous cases,

$$x = \frac{1}{1 + \frac{S}{Cr}} \qquad (34) \qquad\qquad C = \frac{S}{r}\,\frac{x}{1-x} \qquad (35)$$

By equating the horizontal forces on the section, the expression for p becomes

$$p = \frac{C}{S}\,\frac{b'h'}{bd}\left(1 - \frac{h'}{2xd}\right) \qquad (36)$$

Eliminating C and S,

$$p = \frac{b'h'(2xd - h')}{2bd^2r(1 - x)} \qquad (37)$$

From which

$$x = \frac{2bd^2pr + b'h'^2}{2d(bdpr + b'h')} \tag{38}$$

Taking moments about the center of pull in the steel,

$$M_R = \frac{Cb'h'}{6xd}\left(6xd^2 - 3dh'x - 3dh' + 2h'^2\right) \tag{39}$$

or.

$$M_R = \frac{Sb'h'}{6dr}\left(\frac{6xd^2 - 3dh'x - 3dh' + 2h'^2}{1 - x}\right) \tag{40}$$

If the center of the compression in the concrete be assumed to act at the center of the flange, still neglecting the compression in the web below the flange, we have,

$$M_R = \frac{Cb'h'}{4xd}\left(4xd^2 - 2dh'x - 2dh' + h'^2\right) \tag{41}$$

or

$$M_R = \frac{Sb'h'}{4dr}\left(\frac{4xd^2 - 2dh'x - 2dh' + h'^2}{1 - x}\right) \tag{42}$$

or

$$M_R = Spbd\left(d - \frac{h'}{2}\right) \tag{43}$$

Case II. Neutral Axis at Under Side of Flange, $xd = h'$.

The formulas for this case may be obtained either directly or by substituting $\frac{h'}{d}$ for x in the formulas of Case I.

$$C = \frac{Sh'}{r(d - h')} \quad (44) \qquad p = \frac{C}{2S}\,\frac{b'h'}{bd} \quad (45) \quad \text{or} \quad p = \frac{b'h'^2}{2bdr(d - h')} \quad (46)$$

$$M_R = \frac{Cb'h'}{2}\left(d - \frac{h'}{3}\right) \quad (47) \qquad \text{or} \quad M_R = Spbd\left(d - \frac{h'}{3}\right) \quad (48)$$

When the percentage of steel in the beam conforms to that given by equation (46) the value of the moment of resistance may be expressed as follows by substituting in (48) the value of p from (46),

$$M_R = \frac{Sb'h'^2(3d - h')}{6r(d - h')} \tag{49}$$

Case III. Neutral Axis Within the Flange, $xd < h'$.

$$x = \frac{1}{1 + \dfrac{S}{Cr}} \tag{50}$$

$$C = \frac{S}{r}\,\frac{x}{1 - x} \tag{51}$$

$$p = \frac{Cb'x}{2Sb} \tag{52}$$

or

$$p = \frac{x^2 b'}{2br(1 - x)} \tag{53}$$

$$x = \frac{pbr}{b'}\left[\sqrt{\frac{2b'}{pbr} + 1} - 1\right] \tag{54}$$

$$M_R = \frac{Cb'd^2x}{2}\left(1 - \frac{x}{3}\right) = \frac{Cb'd^2}{2\left(1 + \dfrac{S}{Cr}\right)}\left[1 - \frac{1}{3\left(1 + \dfrac{S}{Cr}\right)}\right] \tag{55}$$

or

$$M_R = Spbd^2\left(1 - \frac{x}{3}\right) \tag{56}$$

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